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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

CONTENTS

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover.

WATER-PROOF MASONRY DAMS

By W. Watters Pagon,* M. Am. Soc. C. E.

Synopsis

Until recent years little attention was paid to the possibility that masonry dams might be liable to uplift forces of considerable magnitude, resulting in a dangerous reduction of their factors of safety. When general interest was aroused finally in the subject, the line of attack chosen, save in a few sporadic cases, was to neutralize the uplift by additional masonry. It is the purpose of this paper to draw attention to another method, which has been somewhat foreshadowed by recent designs; namely, that of preventing the uplift, rather than neutralizing it.

Briefly, the method consists in the provision of a water-tight membrane or sheet, placed as near the up-stream face of the dam as is practicable, with in addition a somewhat customary system of under-drainage to ensure the removal of any seepage from the foundation bed. A few suggestive general details are included to illustrate possible means of carrying out this general idea.

A brief history of dam design is given, showing the gradual development of knowledge concerning dams and the reasons for using the several forces that control the profile. As there is so little exact knowledge of the actual internal forces, no hard-and-fast rules can be laid down, but the best modern practice is discussed and a set of formulas of the usual type is stated, actual designs being worked out to determine a specific result.

To determine the amount of saving actually obtainable in specific projects, two dams were designed, one low and one high. Alternate designs for each of these considered, first, that upward pressure would act; and, second, that all uplift would be eliminated in front of a membrane placed 3 ft. more or less from the water face. These designs are shown and the masonry quantities tabulated. Superimposed curves show the relative differences and the saving in masonry.

Brief History of Dam Design

The building of masonry dams has long been practiced, but their rational design has only been established in recent years. Many large masonry dams were built in Spain during the Middle Ages, but often such dams were subject to excessive stresses due to dead weight alone. Modern design dates from about 1853, when de Sazilly, a French engineer, deduced the first general equations for the profiles of dams. His designs were based on the principle of

Note.—Written discussion on this paper will be closed in January, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

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limitation as regards two internal elements: (1) The pressures in the masonry; and (2) the tendency toward shear, or sliding. He also proposed two conditions of exterior loading, namely, with reservoir full and reservoir empty. He limited his allowable pressures to about 6 tons per sq. ft., as compared with modern stresses up to 20 tons; hence his designs were excessively safe. He had observed that no dams had failed because of shear, and, therefore, considered that this did not enter the problem.

Succeeding him another French engineer, Delocre, carried the investigations further in the design of the Furens Dam. He devised a theoretical profile which would meet the same conditions.

Rankine introduced the next important idea. He evolved a theoretical profile that would meet the required pressures economically, limiting the lines of pressure to the middle third of the section. Moreover, since the line of pressure for the full head of water is sharply inclined, whereas that for the reservoir empty is nearly vertical, he proposed that the unit stress at the toe be restricted to a lower value than that at the heel. Similar profiles have been devised more recently.

Modern dam design may be said to date from the Quaker Bridge Dam of the New York Water Supply. This exceeded in height any existing dam by more than 100 ft. Edward Wegmann, M. Am. Soc. C. E., made a careful study of the design, using known principles to devise practical profiles by trial and error. He also increased the allowable pressure to 16 tons, based on a successful Spanish dam which had carried this stress for some centuries. With this increased allowable stress, the most important element then became the position of the line of pressure with reference to the middle third.

Up to this time only the weight of masonry and the horizontal pressure had been considered. It was recognized, however, that masonry is not impervious, but that leakage and damp spots indicated hydrostatic pressure within the structure. Numerous failures emphasized the fact that something was wrong. In the Wachusett Dam uplift was allowed for; moreover the failure of the dam at Austin, Pa., was attributed to the neglect of uplift. The late Frederic P. Stearns, Past-President, Am. Soc. C. E., maintained that uplift should be considered over the entire area of section and at the full hydrostatic value. Mr. Wegmann still maintained that no uplift should be considered, and there were other opinions between these extremes.

In 1912, the late Charles L. Harrison, M. Am. Soc. C. E., proposed* three possible general conditions:

(1).—Contact with a rock bed such as would preclude uplift, and no joints in the masonry.

(2).—Porosity such that the water pressure would be at the full reservoir head at the heel and at tail-race head at the toe, varying uniformly for intermediate points.

(3).—Full hydrostatic head at the heel, and the head of the issuing stream at the toe.

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* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 142.

After considerable discussion it was generally agreed that uplift must be considered, as had already been done in the Wachusett, Olive Bridge, and Kensico Dams. For these structures the uplift was assumed to vary linearly from two-thirds of the hydrostatic head at the heel to zero at the toe. This probably nearly represents present practice. Ice pressure has been variously considered as lying between the crushing strength of ice (47 000 lb. per sq. ft.) and zero. In recent years, the maximum intensity has been frequently used in northern climates.

Quantitative Effect of Uplift on the Size of Dam

A triangular cross-section is theoretically correct for economy, but cannot be entirely adopted in practice. Let, Δ = the ratio of the weight of masonry to that of water per unit volume; H = the depth of water, in feet (approximately the height of masonry above the section); and L = the width of base at the section. Imposing the requirement that the line of pressure follow the middle-third points, it is found that $L = \frac{H}{\sqrt{\Delta}}$ or, if $\Delta = \frac{7}{3}$, $L = 0.654\ H$, provided there is no uplift. When, however, provision is made for uplift by the two-thirds rule, there results, $L = \frac{H}{\sqrt{\Delta - \frac{2}{3}}}$, or $L = 0.775\ H$. For the two cases, the area of the vertical cross-section is $0.327\ H^2$, and $0.3875\ H^2$, respectively, the difference being 15.5% of the larger, or 18.5% of the smaller.

The comparison when ice thrust is considered is not so simple; but assuming a thrust at the top of the dam of 47 000 lb. per lin. ft., there results:

With pressure,

$$L = \sqrt{\frac{4\ 500 + H^2}{\frac{5}{3}}}$$

and without pressure,

$$L = \sqrt{\frac{4\ 500 + H^2}{\frac{7}{3}}}$$

and the ratio of the respective lengths, or areas, is $\sqrt{\frac{5}{7}}$. The saving in masonry will be 15.5% of the larger as before. Hence, no further consideration need be given this factor.

The Water-Proofing Membrane

Apparently, John R. Freeman, Past-President, Am. Soc. C. E., was the first to propose the economy of water-proofing; he suggested* that a thin sheet of lead be built into the dam a few feet from the water face, thus absolutely preventing any uplift within the dam. His idea had reference to the proposed Housatonic Dam, but it was not used because the other engineers considered

* "Report on New York's Water Supply," New York, 1900, p. 265.

that the foundation conditions precluded the possibility of uplift, and they proposed that the masonry be so made as to be water-proof. Hence, the idea was dropped and almost forgotten.

At that time it was thought that masonry could always be made water-tight throughout, but later experience has shown the writer that tightness is by no means certain, so that serious spalling takes place on bridges and dams in cold climates and extensive lime deposits form on the surface in warmer climates, even on structures for which it would be presumed that the concrete was well made. Construction joints, also shrinkage and temperature cracks, may allow some leakage, and thus cause uplift, at least at those sections. Since, therefore, no important dam would be built nowadays without considering uplift, the whole question of water-proofing appears to be important.

Allen Hazen, M. Am. Soc. C. E., has described* a dam in Australia the back of which was built with special care to insure water-tightness. Porous tile pipes were placed parallel to and within a few feet of the back face leading down to a drainage gallery near the base. It was assumed that by collecting the water near the back no uplift could develop beyond that point. The foundation was excellent, but a cut-off wall was built to considerable depth. This case is the only one known to the writer in which provision was made for eliminating uplift and considered in the design.

In the Kensico and Olive Bridge Dams, tile drains and drainage galleries were provided, but were not considered as preventing uplift; in fact, the design was later made of greater weight than was required by computation.

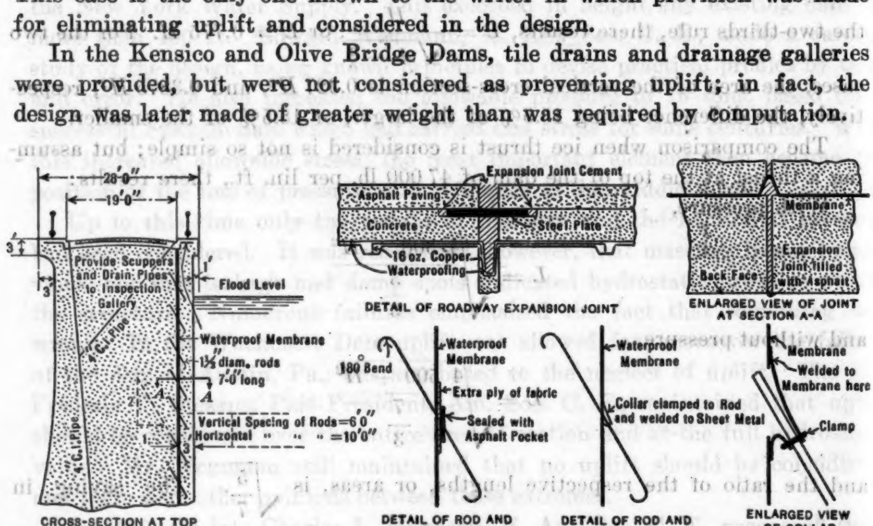

FIG. 1.

Perfectly successful means are available, however, for water-proofing the structure so as to prevent uplift, using a membrane of lead, zinc, copper, or asphalt and cloth. Of necessity any such material must be covered on the up-stream face, hence it is proposed to embed it about 3 ft. from the back, with steps formed in the concrete where possible and inclined tie-rods to ensure that this covering will act as a part of the main body of the dam. Fig. 1 shows clearly how this may be done.

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 154.

Of course, it is assumed that the foundation bed will be tight. If this proves not to be so, then of necessity drainage galleries could not be provided without causing serious leakage; but why build on such a foundation? If the foundation is not entirely tight, it is proposed to carry down the usual cut-off wall, with a water-proofing membrane within it.

When mechanical means are provided for meeting a condition instead of using increased strength or mass, it is good judgment to provide some method of checking from time to time their successful operation. Hence, if such a membrane is installed, drains and galleries should be provided. With such passageways leakage could not cause uplift, but clearly the small cost of water-proofing compared with the saving in masonry and the desirability of keeping water away from all structural concrete both justify the use of such a membrane.

For a low dam the economy of water-proofing will be smaller than for a high dam, because the cost of the membrane varies directly with the height, while the saving in masonry varies as the square of the height. Hence, while in a low dam the cost of the membrane might conceivably be as much as the saving in masonry, it would form only about 3% of the cost for a dam 100 ft. high, as against a saving of masonry of about 12%, or a net saving of 9%; and for a 250-ft. dam the cost would be perhaps 1% against a saving of 14%, or a net saving of 13 per cent.

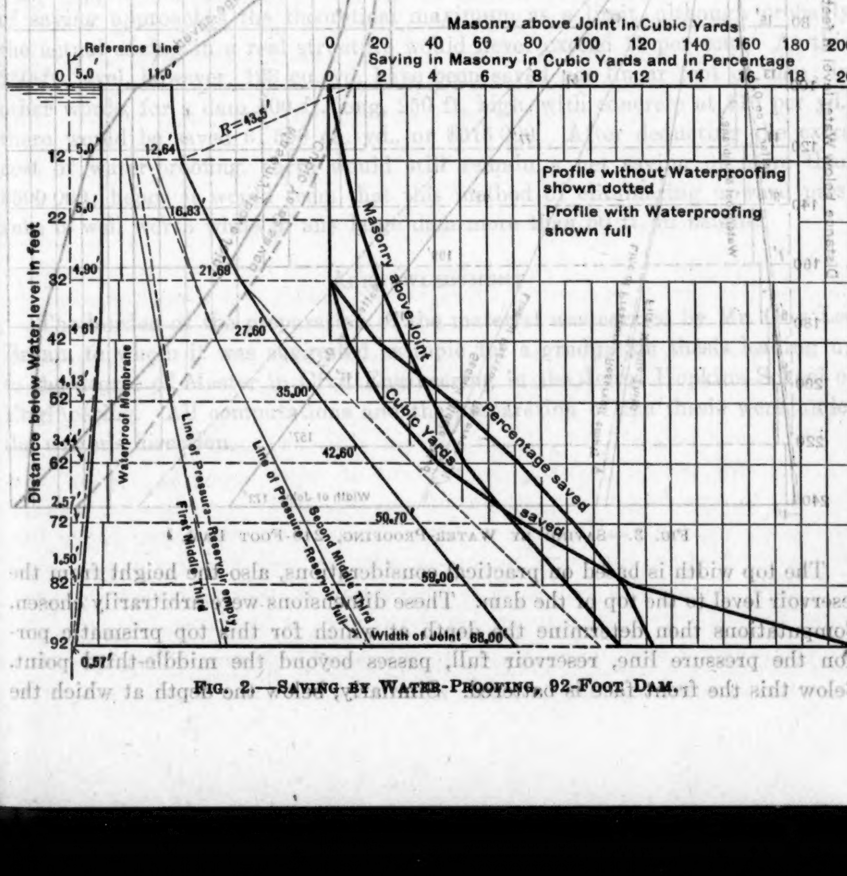

FIG. 2.—SAVING BY WATER-PROOFING, 92-FOOT DAM.

APPLICATION OF THIS IDEA TO TWO SPECIFIC CASES

Thus far the argument has been based on theoretical profiles, but it is now proposed to consider two specific designs, one for a low dam (about 100 ft. high, Fig. 2), the other for a high dam of about 250 ft. (Fig. 3), to determine how near to the theoretical saving it is possible to attain. The savings should approach 15.5% as a limit for increasing height, because at low heights the top width of dam and not the stability is the controlling factor. For each dam a design is made both with and without uplift, and in the latter cases allowance is made for the concrete protection coat over the membrane. The limiting conditions of loading are for the reservoir both full and empty.

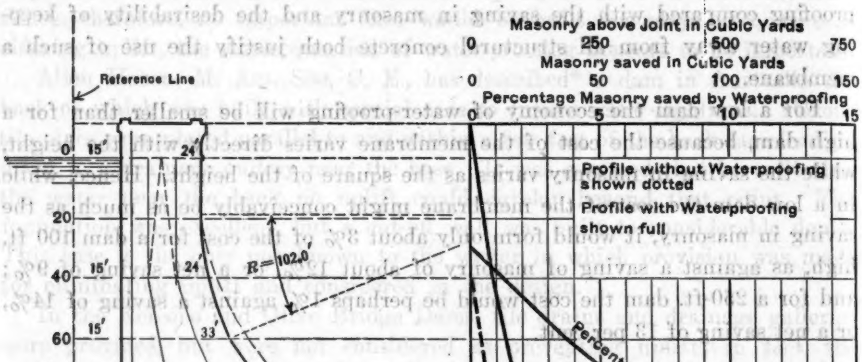

FIG. 3.—SAVING BY WATER-PROOFING, 240-FOOT DAM.

The top width is based on practical considerations, also the height from the reservoir level to the top of the dam. These dimensions were arbitrarily chosen. Computations then determine the depth at which for this top prismatic portion the pressure line, reservoir full, passes beyond the middle-third point. Below this the front face is battered. Similarly, below the depth at which the

pressure line, reservoir empty, passes beyond the inner third point, the up-stream face is battered. Ice pressure has not been considered for reasons previously discussed. Uplift has been assumed to vary from two-thirds of the hydrostatic head at the heel to zero at the toe, although if there were any considerable tail-race depth, the drainage galleries would have to be placed above water to permit inspection, and there might be pressure at the toe greater than zero. The weight of masonry was taken at 146 lb. per cu. ft., making $\Delta = \frac{7}{3}$.

Plotted beside each profile in Figs. 2 and 3 is a diagram showing quantities and savings in masonry. The respective designs are superposed to show the direct comparisons. Clearly the theoretical percentage is being approximated for the greater heights, the saving being 14% for a 250-ft. dam against 15.5% theoretical.

Conclusions

The maximum theoretical saving of masonry is 15.5%, based on a triangular design. The profiles shown contemplate no saving in the top, or parallel, portion because the width of this is fixed by independent conditions. Beginning about 30 ft. below reservoir level, the water-proof dam contains less masonry, and increasingly so, for greater depths. At the 92-ft. depth of the low dam about 12.3% has been saved; and at the 250-ft. section of the high dam, 14.3 per cent. A study of the curves will show that the amount of saving approaches the theoretical maximum as a limit, although probably the actual saving in a real structure would never exceed 15 per cent. At this 250-ft. level, however, 123 cu. yd. have been saved per linear foot of dam; in other words, for a dam 500 ft. long, 250 ft. high, with concrete at $10 per yd., there would be saved 61 500 cu. yd., or $615 000. After deducting the extra cost of water-proofing, there would still remain a net saving of more than $500 000; hence it would seem that this method of eliminating upward pressure is well worth while in any large dam more than 50 ft. in height.

Acknowledgment

The burden of the preparation of the material was carried by Mr. Guy Lee Bryan, to whom it was suggested as topic for a graduation thesis leading up to the degree of Master in Civil Engineering in the Johns Hopkins School of Engineering. All computations and the preparation of the thesis were under the writer's direction.

SOME PROJECTS FOR SEWAGE TREATMENT UNDER THE ILLINOIS SANITARY DISTRICT ACT OF 1917

By Samuel A. Greeley,* M. Am. Soc. C. E.

Synopsis

This paper describes briefly some of the developments under the Illinois Sanitary District Act of 1917. Already fourteen sanitary districts have been organized with populations ranging from 1 700 to 60 000. Several others are in contemplation. Three of the districts have placed sewage treatment works in operation; in five others, major construction work is under way; and, in another, work is expected to start during 1926.

The works in these districts comprise intercepting sewers, pumping stations, settling tanks, and sprinkling filters. In the planning of the works, sewage has been gauged and analyzed, testing stations have been operated, and much interesting experience has been gained through investigation, design, construction, and operation. A variety of conditions have been met, including the treatment of both domestic and industrial sewages.

Costs of construction and operation are of record. Most of these sanitary districts are on streams having little flow during dry seasons so that relatively complete sewage treatment has been required. Such data as are considered useful are recorded herein.

Sanitary District Acts in Illinois

The Sanitary District Act of general application in Illinois was passed by the Legislature and approved by the Governor in 1917 and is designated as the 1917 Act. It was amended in 1919 and 1923.

In addition to the general Act of 1917, there are two other Acts under each of which only one district can be formed, so that they have no further applicability.

The Legislature of 1925 passed the so-called River Conservancy Act making provision for sewage disposal, development of water supplies, and policing. In order to provide a proper transfer of sewerage works from existing authorities (including sanitary districts and municipalities) to the Conservancy District, provisions are contained in the law whereby such transfer can be made upon favorable popular vote so as to compensate existing authorities for such works as have been constructed and such expenses as have been incurred. No

Note.—Written discussion on this paper will be closed in January, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

* (Pearse, Greeley & Hansen), Chicago, Ill.

conservancy districts have as yet been formed, but a petition is contemplated at an early date for a district in the valley of the Fox River.

The Sanitary District of Chicago

The Act creating the Sanitary District of Chicago, passed in 1889, has been amended several times. These amendments have extended the powers of the District and have cured various defects as they developed through administrative experience. One amendment in 1917 provided specifically for the building of sewage treatment works. Sizable annexations have been made from time to time so that at present (1926) the District comprises more than 437 sq. miles, including 50 municipalities.

For operating expenses and fixed charges, the District may levy taxes not to exceed 0.67% of the assessed valuation. The minimum tax levy is not less than $0.18 on $100, for corporate purposes. Since its organization this District, up to 1926, has expended more than $114 000 000 for the construction of the main drainage canal and all the various connecting channels, intercepting sewers, and sewage treatment works. Of this, more than $36 000 000 has been for sewage treatment.

North Shore Sanitary District

The Act of 1911 provides that "whenever any area of continuous territory within the limits of a single county shall contain two or more incorporated cities, towns, or villages owning and operating * * * and procuring a supply of water from Lake Michigan * * * the same may be incorporated as a sanitary district under this Act".

The only area within the State of Illinois which meets this requirement is the strip of land along Lake Michigan extending from the northern limits of the Sanitary District of Chicago to the Wisconsin State line. A part of this area was organized in 1914 as the North Shore Sanitary District. It has since been extended to comprise all the Lake front from the Chicago Sanitary District to the State line. No other sanitary district, therefore, can be organized under the 1911 Act. The salient features of this Act are briefly summarized as follows:

(*a*) Three hundred legal voters may petition the County Judge to submit the proposition of organizing a sanitary district. The County Judge with two judges of the Circuit Court shall constitute a board of commissioners with authority to consider and fix the boundaries of the proposed district to be voted upon.

(*b*) If the district is approved by the voters, its management is vested in a board of five trustees who are appointed by the County Judge and the two Circuit Judges. This commission of judges divides the district into five wards and appoints one trustee for each ward. Each of the trustees is paid $500 per year and they elect a president from among their own number, who receives $1 000 per year. The trustees appoint a chief engineer, superintendent, attorney, and other employees.

(*c*) The duties of the Board of Trustees are to provide for the disposal of the sewage of the district and to preserve the water supplied to its inhabitants from contamination.

(*d*) With popular approval the district is authorized to borrow money and to issue bonds not to exceed 5% of the valuation of taxable property in the district. As this indebtedness is incurred the trustees provide for a direct annual tax sufficient to pay the interest and to discharge the principal of the indebtedness within at least twenty years. In addition, the Board of Trustees may collect taxes not to exceed 0.33% of the value of the taxable property.

Up to 1926, this District has issued no bonds, but has built all its works, which have cost so far upwards of $350 000, from accumulated taxes. Recently an amendment has been sought providing for the right to construct works by special assessment in general accordance with the Act of 1897 concerning local improvements, but this amendment failed. It is interesting to note that this Board of Trustees is authorized to enter into contract with any city or village for the disposal of garbage.

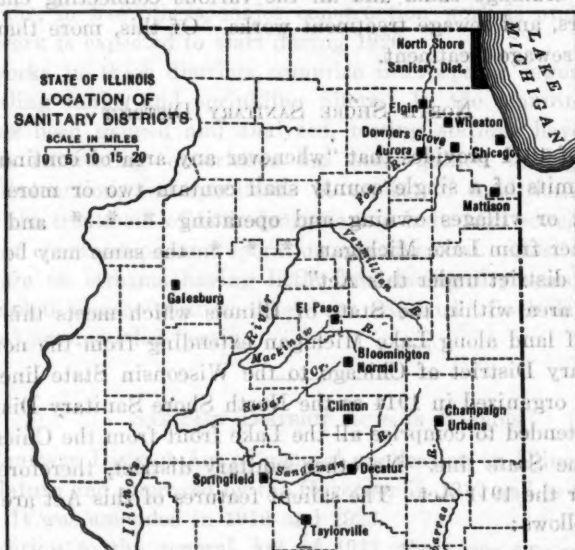

FIG. 1.

ACT OF 1917

The first district organized under the Act of 1917, in Decatur, Ill., was approved by popular election in August, 1917. Since then thirteen other districts have been similarly organized. Including the Sanitary District of Chicago and the North Shore Sanitary District, there are, therefore, fifteen districts in the State (Fig. 1). The salient features of this Act are briefly summarized as follows:

(*a*) Whenever contiguous territory containing one or more incorporated communities shall be so situated that the construction of a plant or plants

for sewage treatment will conduce to the preservation of the public health, this territory may be incorporated as a sanitary district. No territory located more than three miles from the limits of a city, town, or village can be included under such a district.

(*b*) One hundred legal voters may petition the County Judge of the County in which the major portion of the proposed district is located to submit the proposition of organizing a sanitary district. The County Judge and the two Judges of the Circuit Court consider and fix the boundaries of the proposed district, after giving opportunity for discussion in public meeting.

(*c*) If the district is approved by the voters its management is vested in a board of three trustees, appointed by the County Judge and residents of the district. These trustees elect one of their number as president and appoint the necessary officers. The salaries of the trustees are limited to $100 per year each.

(*d*) The duties of the Board of Trustees are to provide for the disposal of the sewage of the District and to preserve the water supplied to its inhabitants from contamination.

(*e*) The District is authorized with popular approval to borrow money and to issue bonds not to exceed 5% of the valuation of taxable property. As this indebtedness is incurred the trustees provide for a direct annual tax sufficient to pay the interest and to discharge the principal of the indebtedness within at least twenty years. The Board of Trustees may also levy and collect annual taxes not to exceed 0.5% of the value of the taxable property. In addition, the Board may levy and collect a like sum, provided this additional annual tax has been authorized by the voters.

(*f*) The trustees are given power to prevent the pollution of any waters from which a water supply may be obtained by any community in the District within a radius of fifteen miles from the intake of the water supply.

(*g*) The 1923 Amendment to this Act gives the trustees authority to pay for the construction of sewers, drains, laterals, and appurtenances by special assessment in general accordance with the Act of 1897 concerning local improvements.

Up to 1924 bonds had been voted for the construction of sewage treatment projects in nine of the thirteen districts. Up to 1926 the total bonds so authorized has amounted to $4 504 000. In addition, some money has been accumulated for construction purposes from annual taxes.

The data shown in Table 1 indicate that the amount of bonds allowed Illinois Sanitary Districts is in general not sufficient to meet the cost of intercepting sewers and sewage treatment works. At Decatur, where the cost of sewage disposal works is increased by the industrial sewages, the amount of money available from bonds has been but little more than one-half the total necessary expense. The difference has been secured by accumulating the surplus from annual taxes.

General statistics of a number of the Illinois Sanitary Districts are shown in Table 2. The general location of the sewage treatment projects of the North Shore Sanitary District is shown in Fig. 2.

TABLE 1.—SANITARY DISTRICTS OF ILLINOIS. FINANCIAL DATA.

District.	Year of data.	Approximate population.	Assessed valuation.	Bond Limit.		Bonds authorized.	Approximate Construction Cost.		Construction cost per capita.
				Total.	Per capita.		Sewers and appurtenances.	Sewage treatment works.	
North Shore	1925	58 000	$28 952 020	$1 447 601	$25.00	None			
Decatur	1923	48 000	17 791 937	889 597	18.50	$860 000	$752 848*	$569 904*	$27.60
Bloomington-Normal	1925	36 000	16 528 549	826 427	23.00	700 000	144 178*	630 000†	20.60
El Paso	1920	1 638				43 000			
Urbana-Champaign	1926	40 000	13 000 000	650 000	16.25	500 000	89 500*	397 884*	12.20
Downers Grove	1921	3 540	1 500 000	75 000	21.20	75 000	15 000*	50 000*	18.85
Elgin	1925	30 000	14 475 128	723 757	24.10	700 000	268 500*	435 210*	23.40
Taylorville	1926	10 000	2 523 504	126 175	12.62	126 000	61 445*	30 940*	9.24
Springfield	1924	60 000	32 656 746	1 632 838	27.21	1 500 000	1 603 000*	525 000†	35.50
Galesburg	1923	25 000	12 000 000	600 000	24.00	None	450 000†	100 000†	22.00
Clinton	1926	7 500	2 311 200	130 000	17.30	None			
Aurora	1926	41 000	20 000 000	1 000 000	24.40	None			

* Contracts.
† Estimated.

TABLE 2.—SANITARY DISTRICTS OF ILLINOIS. GENERAL DATA.

District.	Year formed.	Area included, in square miles.	Population. 1920 Census.	Population. Treatment plant, capacity.	Population. Intercepting sewer, capacity.	Population per Acre. 1920 Census.	Population per Acre. Treatment plant, capacity.	Population per Acre. Intercepting sewer, capacity.
North Shore	1914	25.0	36 000	......	....	2.3	....	....
Decatur	1917	33.0	43 818	60 000	120 000	2.1	2.8	5.7
Bloomington-Normal	1919	8.3	33 868	50 000	75 000	6.4	9.4	14.1
El Paso	1919	2.3	1 638	......	....	1.1	....	....
Urbana-Champaign	1921	8.5	35 000	45 000	70 000	6.4	8.3	12.8
Downers Grove	1921	5.6	3 543	5 000	25 000	1.0	1.4	7.0
Elgin	1922	8.3	28 260	37 500	75 000	5.3	7.0	14.0
Taylorville	1923	3.5	7 000	4 500	....	3.1	....	....
Springfield	1924	36.2	63 000	90 000	193 000	2.7	3.9	8.3
Galesburg	1924	17.2	23 834	30 000	70 000	2.2	2.7	6.4
Wheaton	1925	15.5	4 137	10 000	....	.4	1.0	....
Clinton	1925	1.6	5 898	......	....	5.9	....	....
Aurora	1925	11.6	36 397	......	....	4.9	....	....
Matteson	1925	....	......	......	....	....	....	....
Hinsdale	1926	....	......	......	....	....	....	....

SEWAGE QUANTITIES

In many of the districts, weirs have been built at the end of one or more main sewers for measuring the sewage flow. Table 3 gives the results of gaugings at Springfield in 1923. The Cook Street Sewer has a drainage area of 916 acres and an estimated population of 8 710 (1923). Gaugings were made following rainy weather and dry weather, the average sewage flow increasing from 89 to 148 gal. per capita per 24 hours during the wet season. Wet-season flows appear to be considerably higher than dry-season flows in the Illinois Sanitary Districts.

TABLE 3.—SPRINGFIELD, ILLINOIS, MEASURED SEWAGE FLOW. SUMMARY TABLE.

Sewer.	Dates.	Sewage Flow Rates. Average per capita per day.	Maximum Hour. Per capita per day.	Maximum Hour. Percentage of average.	Minimum Hour. Per capita per day.	Minimum Hour. Percentage of average.
Ridgeley*	October 9-11	82	114	139	62	76
Amos Branch†	October 12-14	42.5	127	300	20	47
Town Branch§	October 20-23	155	217	140	112	72
Cook Street (wet weather)‡	October 23-25	148	176	119	125	85
Cook Street (dry weather)*	November 18-19	89	103	116	77	87
Oak Knowles§	October 26-28	149	153	103	140	94

* Dry-weather flow.

† Some rain during gaugings—dry flow.

‡ Wet-weather flow.

§ Sewers located in natural valley. Flows probably affected by sub-surface drainage from large area.

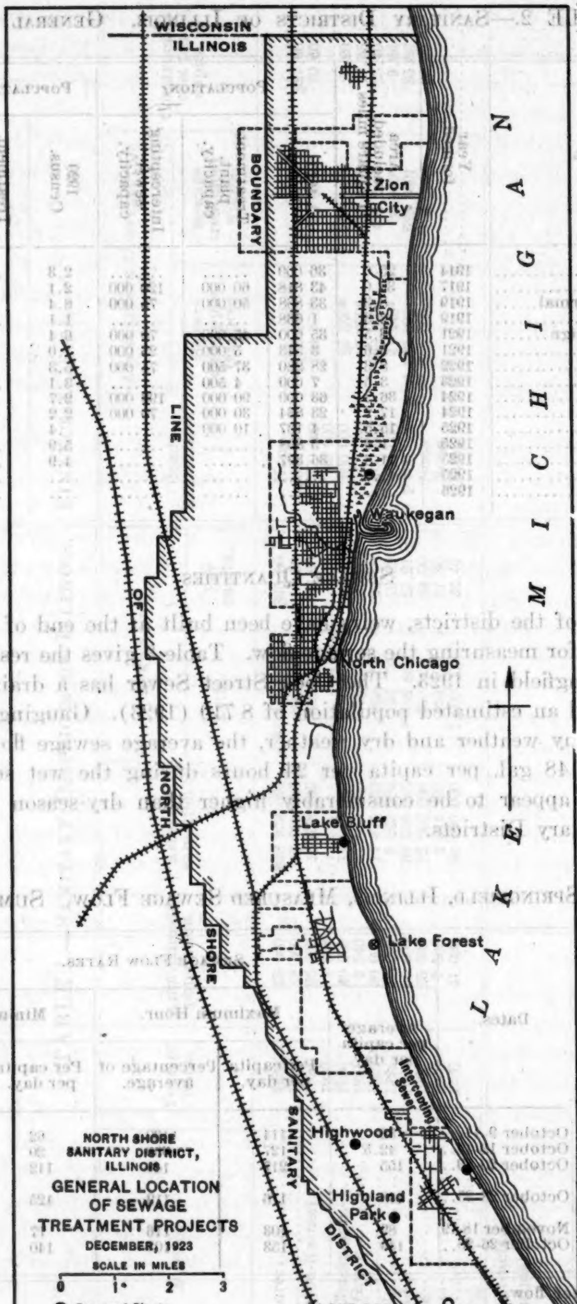
WISCONSIN
ILLINOIS
BOUNDARY
LINE
OF
NORTH
SHORE
SANITARY
DISTRICT
Zion
City
Waukegan
North Chicago
Lake Bluff
Lake Forest
Highwood
Highland
Park
Intercepting
Sewer
L A K E M I C H I G A N
LAKE CO.
COOK CO.
NORTH SHORE
SANITARY DISTRICT,
ILLINOIS
GENERAL LOCATION
OF SEWAGE
TREATMENT PROJECTS
DECEMBER, 1923
SCALE IN MILES
0
1
2
3
Proposed Plants
Remodelled Plants
Plants built

FIG. 2.

At Urbana-Champaign, the State Water Survey maintained a weir at the end of the Champaign Sewer for almost a year. Fig. 3 shows the rates of sewage flow following rains. The lower broken line indicates typical dry-

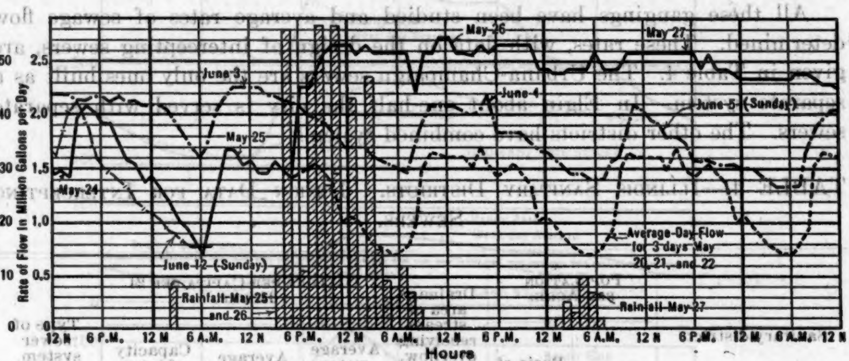

FIG. 3.—EFFECT OF RAIN ON SEWAGE FLOW, CHAMPAIGN OUTLET SEWER, DURING 1921.

season hourly rates of flow. The upper lines show the rates following (not during) rains. It took from May 27 to June 9 (13 days) for the sewage flow to reach normal dry-weather rates. This system was built as separate sewers, but appears to receive some surface sewage. Typical wet and dry-season hourly rates of sewage flow and also of the water pumpage at Champaign are shown in Fig. 4.

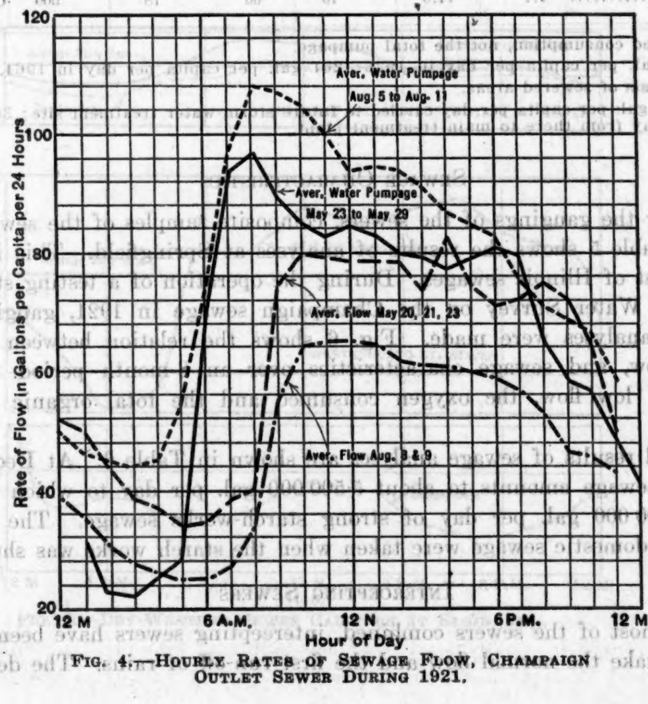

FIG. 4.—HOURLY RATES OF SEWAGE FLOW, CHAMPAIGN OUTLET SEWER DURING 1921.

Fig. 5 shows hourly rates of sewage flow as gauged at Elgin. All the per capita rates are quite low except the West Chicago Street Sewer, which receives the sewage from a malted milk factory, totalling about 650 000 gal. per day.

All these gaugings have been studied and average rates of sewage flow determined. These rates, with data on the design of intercepting sewers, are given in Table 4. The Urbana-Champaign sewers are the only ones built as a separate system. In Elgin about one-half the city is served with separate sewers. The other districts have combined systems.

TABLE 4.—ILLINOIS SANITARY DISTRICTS. DESIGN DATA FOR INTERCEPTING SEWERS.

Sanitary district.	Population per Acre.		Drainage area of stream receiving overflow, in square miles.	Gallons per Capita per 24 Hours.			Type of sewer system.
	Present.	Basis of design.		Average water consumption.	Average sewage flow.	Capacity of intercepting sewer.	
Decatur	2.1	5.7	862	105*	125	375	Combined
Urbana-Champaign	6.4	12.8	76	40	75	136	Separate
Elgin	5.3	13.0	1 520	40	100	261–620†	Half Combined Half Separate
Springfield:							
West Side	8.7 (‡)	17.0‡	265	71	125	425	Combined
East Side	8.7 (‡)	17.0‡	110	71	125	2 125–300§	Combined
Galesburg	4.1	14.0	10	66	78	500	Combined

* Metered consumption, not the total pumpage.

† 620 gal. per capita per day in 1926; 261 gal. per capita per day in 1964.

‡ On basis of sewered areas.

§ 2 125 gal. per capita per day carried to future storm-water treatment site; 300 gal. per capita per day from there to main treatment plant.

SEWAGE CHARACTERISTICS

During the gaugings of the sewers, composite samples of the sewage were taken. Table 5 shows the results of analyses at Springfield. This is one of the weakest of Illinois sewages. During the operation of a testing station by the State Water Survey on the Champaign sewage in 1921, gaugings and chemical analyses were made. Fig. 6 shows the relation between rainfall, sewage flow, and sewage characteristics over an 8-month period. During periods of low flow, the oxygen consumed and the total organic nitrogen increased.

Typical results of sewage analyses are shown in Table 6. At Decatur the domestic sewage amounts to about 5 500 000 gal. per day to which is added about 4 500 000 gal. per day of strong starch-works sewage. The analyses typical of domestic sewage were taken when the starch works was shut down.

INTERCEPTING SEWERS

With most of the sewers combined, intercepting sewers have been proportioned to take the normal flow and the first run-off of rains. The design has

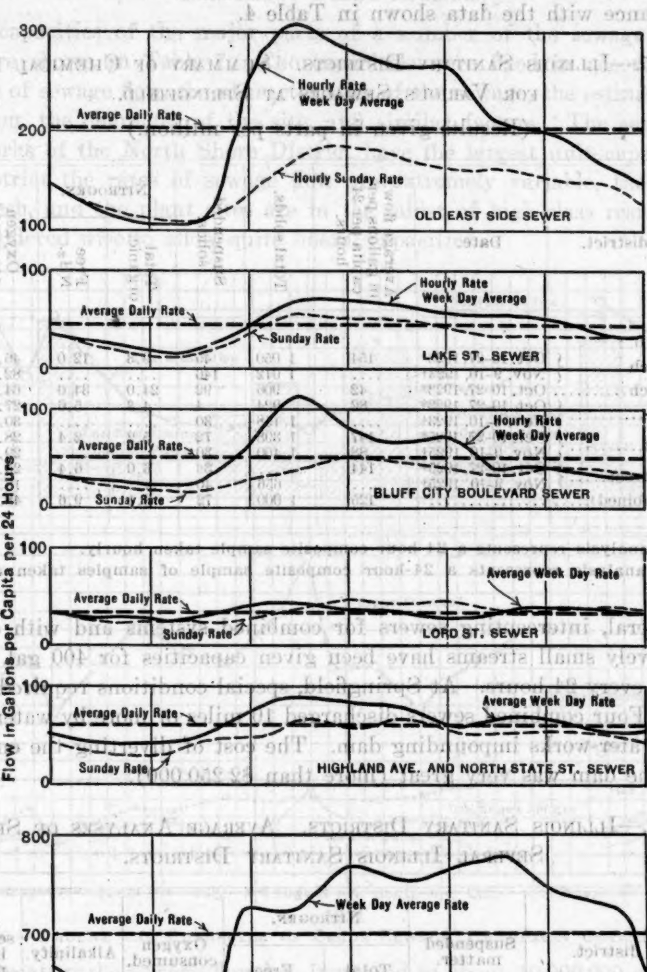

FIG. 5.—DRY-WEATHER SEWER GAUGINGS AT ELGIN, ILL.

been based on a per capita sewage flow from the future estimated population in accordance with the data shown in Table 4.

TABLE 5.—ILLINOIS SANITARY DISTRICTS. SUMMARY OF CHEMICAL ANALYSES FOR VARIOUS SEWAGES AT SPRINGFIELD.

(Results given in parts per million.)

Sanitary district.	Date.	Average flow in gallons per capita per 24 hours.	Total solids.	Suspended solids.	Nitrogen. Total organic.	Nitrogen. Free NH_3.	Oxygen consumed.	Alkalinity as $CaCO_3$.
Springfield, Ill.:								
Town Branch........	Oct. 10-27, 1923*	154	1 050	46	9.3	12.0	46.0	350
	Nov. 9-10, 1923†	....	1 012	146	....	....	82.0	....
Amos Branch.........	Oct. 10-27, 1923*	42	906	92	24.0	34.0	64.2	412
Ridgely.............	Oct. 10-27, 1923*	82	924	4	4.3	5.6	27.2	330
	Nov. 9-10, 1923†	....	1 158	30	....	....	30.6	....
Cook Street.........	Oct. 10-27, 1925*	147	1 398	72	5.2	2.4	28.8	340
	Nov. 9-10, 1925†	88	1 400	30	....	....	29.2	....
Governor Street.....	Oct. 10-27, 1925*	144	......	36	6.0	6.4	25.7	317
	Nov. 9-10, 1925†	....	656	40	....	....	18.0	....
Typical (Combined)....	....	125	1 000	72	8.1	9.6	48.0	344

* Each analysis represents a 24-hour composite sample taken hourly.

† Each analysis represents a 24-hour composite sample of samples taken at 30-min. intervals.

In general, intercepting sewers for combined systems and with overflows into relatively small streams have been given capacities for 400 gal. or more per capita every 24 hours. At Springfield, special conditions required a larger capacity. Four combined sewers discharged 10 miles or more by water above a proposed water-works impounding dam. The cost of diverting the entire flow to below the dam was very great (more than $2 250 000).

TABLE 6.—ILLINOIS SANITARY DISTRICTS. AVERAGE ANALYSES OF SEWAGE IN SEVERAL ILLINOIS SANITARY DISTRICTS.

Sanitary district.	Suspended matter.	Nitrogen. Total organic.	Nitrogen. Free ammonia.	Oxygen consumed.	Alkalinity.	Average sewage flow, in gallons per capita per 24 hours.
Decatur: Domestic, 1924...	240	9.6	18.4	61	368	125
Decatur: Combined, 1924*.	291	77	17.0	380	...	200
Springfield..................	72	8.1	9.6	48	344	125
Urbana-Champaign	...	12.3	16.4	59	420	75
Elgin........................	296	29.0	23.0	149	532	100

* Varies with operations at starch works.

It was decided to intercept 2 125 gal. per capita per 24 hours from these sewers and to bring this quantity to one point for treatment. The first flow up to 300 gal. per capita per 24 hours will be pumped to a point below the proposed dam and there passed through complete treatment works. The overflow will be chlorinated near the pumping station. In some other cases additional capacity has been allowed for special industrial sewages.

Treatment Plant Loadings

The capacities of the major parts of a number of the sewage treatment plants are shown in Table 7. The capacities were fixed in accordance with the rates of sewage flow, the characteristics of the sewage, the estimated future population, the character of the site, and similar factors. The sewage treatment works of the North Shore District have the largest unit capacities. In that District the rates of sewage flow are extremely variable, the sewage is quite fresh, and the plant sites are in the midst of high-class residences. It was considered wise to allow quite liberal capacities.

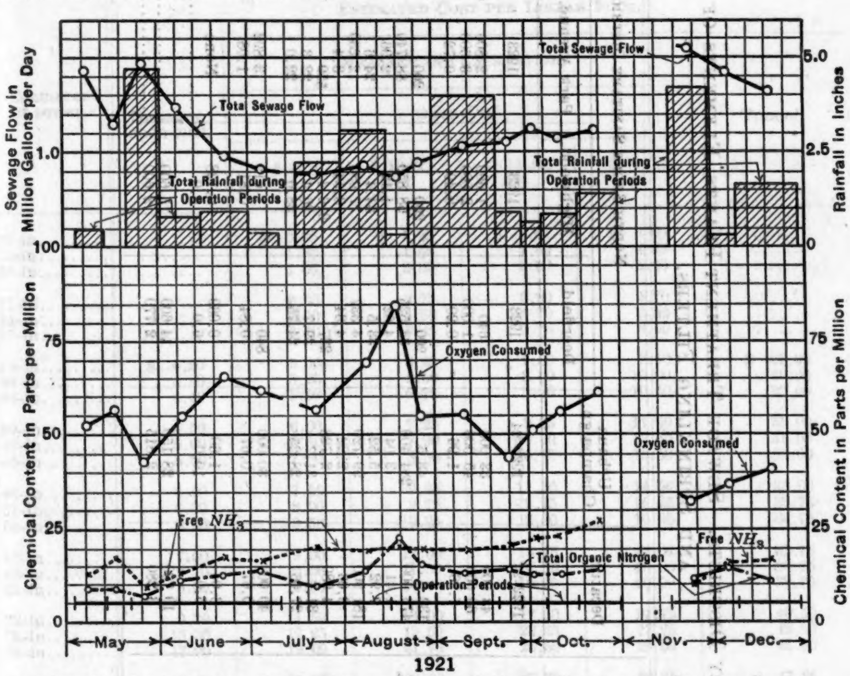

Fig. 6.—Amount and Character of Crude Sewage, Champaign Outlet Sewer.

At Decatur, the sewage flow has increased to about 10 000 000 gal. per day, or about 25% more than the rated capacity of the plant. At this rate of flow, the displacement period in the Imhoff tanks is about 1.15 hours. However, during more than a year of operation, these tanks have reduced the suspended matter by about 75% by weight, delivering an effluent with an average of 70 parts per million of suspended matter. The rate of sewage flow through the plant at Decatur is relatively uniform throughout the 24 hours, ranging from 8 000 000 to 11 000 000 gal. per day. The strength of the sewage adds a further burden on the sprinkling filter which is to be handled through the construction in 1927 of a pre-aeration plant now under design.

Construction Costs

Many miles of intercepting sewers have been built under a wide variety of construction conditions ranging from tunnel work in rock or water-bearing

TABLE 7.—ILLINOIS SANITARY DISTRICTS. SEWAGE TREATMENT PLANTS, ELEMENTS OF IMHOFF TANKS, AND SPRINKLING FILTERS.

Elements of design.	Downers Grove.	Decatur.	Urbana-Champaign.	North Shore Sanitary District.				Elgin, from design plans.
				Deerfield.	Highwood.	Park Avenue.	Lake Bluff.	
Year built	1921	1922-24	1923-24	1923	1923	1923	1923	1925-26
Population:								
Actual	3 550	48 000	33 000	640	1 500	2 800	700	28 500
Basis of design	5 000	60 000	45 000	1 000	2 000	3 600	1 400	87 500
Sewage Flows, in 1 000 000 per day:	......	7.96	1.96	0.20	0.40	0.72	0.28	3.31
Design basis, gallons per capita per day	......	132.5	43.5	200	200	200	200	88
Settling Capacity, in Gallons:	45 050	476 000	284 400	33 525	68 850	88 740	46 820	411 200
Hours (actual)	......	1.41	3.04	4.02	4.12	2.96	4.0	2.99
Gallons per capita	9.01	7.92	5.52	33.5	34.4	24.6	33.4	11.0
Sludge Capacity, in Cubic Feet:	10 000	181 900	99 720	4 337	7 084	12 230	4 688	80 400
Per capita	2.0	3.03	2.22	4.34	3.54	3.4	3.35	2.15
Gas Vent Area, in Square Feet:	132	4 980	2 728	237	316.4	559	300	1 980
Percentage of tank area	17.2	31.1	31.7	29.7	26.4	22.3	29.7	24.2
Maximum water depth, in feet	31.6	27.06	28.25	24.75	28.0	22.0	25.5	29.5
Sludge Bed Area:								
Total square feet	2 500	40 000	30 000	840	1 612	3 892	1 585	24 750
Per capita	0.50	0.67	0.67	0.84	0.81	1.08	1.13	0.67
Sprinkling Filter:								
Acres	0.268	3.04	1.60	0.069	0.128	None	None	1.54
Depth, in feet	7.25	6.0	10.0	6.0	6.0	....	....	8.0
Population Loading:								
Per acre	18 650	19 700	28 150	14 600	15 600	....	....	24 400
Per acre-foot	2 572	3 280	2 815	2 770	2 600	....	....	3 050

gravel to open-cut work along the Fox River at Elgin. Space does not permit a complete statement regarding unit costs for sewer construction. The most recent bids received at Springfield for intercepting sewer work aggregated about $1 500 000. These bids have been quite close to the estimates in the preliminary report. Table 8 shows these estimated costs of sewers of different

TABLE 8.—Springfield, Illinois, Estimated Unit Construction Cost of Sewers.

Diameter of sewer.	Estimated Cost per Linear Foot.					
	Masonry and Excavation.					
	Masonry (open cut).	Open Cut.				Tunnel.
		0–6-ft. cut.	6–12-ft. cut.	12–18-ft. cut.	18–24-ft. cut.	
12-in........	....	$ 0.95	$ 1.25	$ 2.00	$ 3.00	....
15-in........	....	1.30	1.65	2.60	3.20	....
18-in........	....	1.65	2.05	3.00	4.40	....
21-in........	....	2.30	2.75	3.90	5.70	....
24-in........	....	2.90	3.50	4.70	6.80	....
27-in........	....	3.70	4.45	5.70	8.50	....
30-in........	$ 4.20	5.00	6.25	7.50	9.50	$ 27.00
33-in........	4.30	5.25	6.50	8.00	10.00	28.00
36-in........	4.40	5.50	6.75	8.50	10.50	29.00
39-in........	4.50	6.00	7.25	9.25	11.50	30.00
42-in........	5.00	6.50	7.90	10.00	12.50	31.00
45-in........	5.50	7.00	8.50	10.75	13.50	32.00
48-in........	6.00	7.50	9.25	11.50	14.50	33.00
51-in........	6.30	8.00	9.75	12.25	15.25	33.75
54-in........	6.60	8.50	10.25	13.00	16.00	34.75
57-in........	7.00	9.00	11.00	13.75	17.00	35.75
60-in........	7.70	9.50	11.50	14.50	18.00	36.75
66-in........	8.50	10.50	13.00	16.00	20.00	38.75
72-in........	9.30	11.50	14.00	17.50	21.75	40.50
78-in........	15.00	17.25	19.50	23.25	27.25	42.50
84-in........	17.00	19.00	21.75	25.50	29.50	45.00
90-in........	19.00	21.25	24.00	27.75	32.25	47.25
96-in........	21.00	23.50	26.50	30.00	35.00	49.50
9-ft........	27.00	29.00	32.00	36.00	41.50	55.00
10-ft........	32.00	35.00	38.00	42.50	49.00	62.00
11-ft........	38.00	41.00	45.00	50.00	56.50	69.50
12-ft........	44.00	47.50	52.00	57.50	64.50	78.00
13-ft........	50.00	54.50	59.00	65.00	72.50	86.50
14-ft........	58.00	62.00	66.00	73.00	81.00	95.00
15-ft........	64.00	68.00	73.50	80.00	89.00	104.00
16-ft........	70.00	75.00	80.00	87.50	96.50	112.00
17-ft........	77.50	82.50	88.50	96.00	105.00	121.00
18-ft........	86.00	91.00	97.50	105.50	115.50	129.00
19-ft........	95.00	100.00	106.50	115.00	125.50	137.50
20-ft........	100.00	105.50	112.50	122.00	133.00	146.00

sizes for different depths of cut. To these unit prices additional cost items were added for manholes, railroad crossings, rock excavation, etc. More recent bids have been lower.

TABLE 9.—ILLINOIS SANITARY DISTRICTS. TYPICAL UNIT COSTS OF SEWAGE TREATMENT WORKS.

District	Year.	Basis Design.		Cost per Million Gallons per Day.					Cost per Capita.				
		Population.	Average flow, in million gallons per day.	A.	B.	C.	D.	E.	A.	B.	C.	D.	E.
Decatur	1923	60 000	7.96	....	$20 400	$41 400	$9 600	$71 400	....	$2.70	$5.50	$1.26	$9.46
Urbana-Champaign	1923	45 000	1.96	$21 000	66 000	126 000	5 260	218 000	$0.92	2.86	5.50	0.23	9.51
Elgin	1925	37 500	3.31	25 200	33 100	62 600	17 100	128 000	2.22	2.92	5.52	1.51	12.17
Downers Grove	1922	5 000	0.50	12 000	25 000	58 000	5 000	100 000	1.20	2.50	5.80	0.50	10.00
Wheaton	1925	10 000	1.00	....	....	....	....	134 000	....	....	....	....	13.40

NOTE.—(A), Pumping stations; (B), Imhoff tanks, sludge beds, and appurtenances; (C), sprinkling filters, secondary tanks, and appurtenances; (D), grit basins, laboratory buildings, and miscellaneous items; and (E), total.

Table 9 shows unit costs of construction for several completed projects. These costs should be scrutinized in conjunction with the treatment plant loadings of Table 7. The unit figures are computed from the bids. The construction work in each case was good and the contractors' prices were fair. Extras at the finished plants at Decatur and Urbana amounted to about 0.3 per cent. Costs for complete treatment works, including Imhoff tanks, sprinkling filters, and appurtenances (but without pumping stations), are indicated as about $10 per capita.

Sprinkling Filter Stones

Illinois and Indiana limestones have been used for the sprinkling filters. The selection was based on cost and on a careful study of experience with similar stone in operating filters. A record of the chemical analyses of several of these filter stones is given in Table 10.

TABLE 10.—Illinois Sanitary Districts. Analysis of Sprinkling Filter Stones.

Item.	Decatur.*	Deerfield Avenue Plant. North Side Sanitary District.†	Urbana-Champaign.‡	Elgin.§
Calcium carbonate	51.93%	54.24%	96.49%	46.71%
Magnesium carbonate	37.81%	42.78%	1.47%	38.95%
Silica	4.08%	0.68%	1.60%	11.78%
Alumina	5.16%	2.23%	......	1.82%
Iron	0.43%	Trace	......	0.45%
Iron and alumina	......	......	0.38%	......
Sulfur	......	......	......	0.17%
Lime (calcium sulphate)	......	......	......	0.06%
Totals	99.41%	99.93%	99.94%	99.94%

* Material from Lehigh Stone Company, Kankakee, Ill., analysis given in average of two analyses.

† Average of analysis of two samples.

‡ Analysis of stone from Greencastle, Ind., quarry of Ohio-Indiana Stone Company. Part of the filter stone used was obtained from the Lehigh Stone Company, Kankakee, Ill.

§ Average of analysis of three samples.

The stone has been delivered to the filters by truck or by industrial car. After screening to size at the quarry, the material has been screened again at the site of the work to remove small particles and stone dust. If the stone was wet so that too much dust adhered to the surface, it has been washed. Approximate figures for the cost of filter stone in place are given in Table 11.

TABLE 11.—Approximate Cost of Filter Stone in Place.

District.	Cost per Cubic Yard. Material.	Placing.	Total.
Decatur	.....	.....	$4.06
Urbana-Champaign	.....	.....	2.80
Elgin	$2.40	$1.10	3.50
Downers Grove	3.00	0.65	3.65

The bulk of the stone has been between 1 in. and 2 in. in size. Larger stone up to 4¾ in. is used for the bottom 6 in. of the filters.

Treatment Plant Site Areas and Locations

Some data on the area and location of treatment plant sites may be of interest, as shown in Table 12. As available funds have permitted, work has been undertaken to improve the general appearance of the grounds about the treatment works through planting, roadways, ornamental lights, etc. As already noted, the treatment plants of the North Shore District are, in some instances, within 350 ft. of first-class residences. The others are not so critically located.

TABLE 12.—Illinois Sanitary Districts. Area and Location of Treatment Plant Sites.

District.	Distance from approximate center of city, in miles.	Area of plant site, in acres.	Direction of plant site from center of city.
Decatur	2.3	57	W.S.W.
Urbana-Champaign	1.4	51	E.N.E.
Elgin	1.3	39	S.
Downers Grove	1.5	7.5	W.
Springfield	3.0	....	N.
Wheaton	2.2	8.4	S.W.

Operating Results

A study of operating results is beyond the scope of this paper. The plants at Decatur and Urbana-Champaign have fully equipped laboratories and daily routine chemical analyses are made. In the large districts, the best of operation has been secured. In fact the disposition of the District Trustees, to place plant operation in the hands of skilled and competent operators, constitutes one of the chief merits of the Illinois Sanitary District Act. This is reflected, not only in the character of Plant Superintendents, but also in the association of Sanitary District Trustees, through which an earnest effort is made to further improve the results of the work done by the various districts. An annual meeting is held for the discussion of financial, construction and operating problems by the Trustees with their Superintendents and Engineers. Thus, successful progress and method in one District is made available to the Trustees of the other Districts.

The annual budget for the operation of the sewage treatment works at Decatur is itemized as follows:

1925-26 Budget

Payroll:	
Plant foreman and three men	$6 000
Extra summer help, three men	3 000
Supervision	4 000

Brought forward	$18 000
Plant equipment and maintenance	2 000
Electric Power:	
Main Street Pump Station	1 500
Sewage Treatment Plant	1 500
Office supplies and printing	125
Laboratory equipment	250
Automobile	800
Landscaping	1 000
Gasoline and oil	300
Recording gauge	150
Contingencies	1 555
Total	$22 180

Table 13 gives typical monthly chemical analyses for two of the plants.

TABLE 13.—ILLINOIS SANITARY DISTRICTS. GENERAL ANALYTICAL DATA, SEWAGE TREATMENT PLANT OPERATION.

	Parts per Million.			
	Urbana-Champaign.		Decatur.	
	Raw sewage.	Final effluent.	Raw sewage.	Final effluent.
Suspended solids	130	......	260	40
Oxygen consumed	......	36.6	200	35
Ammonia nitrogen	12	13.5	32	......
Organic nitrogen	10	6.4		
Nitrate nitrogen	1.26	9.4	....	10-12
Nitrite nitrogen	1.60	4.56		
Biological oxygen demand	......	......	500-700	30-40

ACKNOWLEDGMENTS

Much of the information contained in this paper has been furnished by the trustees and officers of the sanitary districts. Helpful assistance has been given by principals and assistants in the writer's office, especially Messrs. R. S. Rankin and Paul E. Langdon, to whom grateful acknowledgment is made.

EXPERIMENTAL DEFORMATION OF A CYLINDRICAL ARCHED DAM

By B. A. Smith,* M. Am. Soc. C. E.

Synopsis

Some experimental confirmation of the formulas given in the writer's paper on "Arched Dams"† seemed to be desirable. With this object in view, a cylinder of india rubber was prepared having removable wooden false bottoms. One of these false bottoms was made conical in form, so that when the cylinder was bound to it, the flare would correspond to zero bending moment at the base; this therefore would represent the case of the base simply supported. The other false bottom was made cylindrical so that when the india rubber cylinder was bound to it, the sides at the base would be vertical, corresponding to the practical case of the base encastrée.

A test specimen of the same kind of india rubber was prepared for the purpose of determining its Young's modulus. A direct determination was also made of this constant using the cylinder itself; the value thus determined differed somewhat from the values obtained from the test specimen.

The actual observations of the deformation lie between the values calculated from Young's modulus as determined from the test specimen and from the cylinder itself. In making the observations measurements were first taken with the cylinder empty, and then filled with mercury. Nothing unusual was observed in the case of the base simply supported, but in the case of the base encastrée, there was an abrupt change of direction at the base. A repetition of this experiment, carried out subsequently, in which the displacements near the base were measured inside instead of outside, showed that the shape of the inside surface agreed with the calculated shape.

Determination of the Elastic Constants

A special test piece of the same composition as the cylinder itself was prepared for the purpose of determining the Young's modulus of the india rubber. Its bulk modulus was assumed to be so large that the volume could be treated as constant during the measurements.

The test piece was 6.3 in. (16.0 cm.) long, 0.51 in. (1.30 cm.) wide, and 0.49 in. (1.25 cm.) thick. Tested by direct stretching, a length of 5 in. elongated as follows:

Load, in pounds.	Length, in inches.
0	5.00
2	5.28
4	5.65

* Civ. and Hydr. Engr., Melbourne, Victoria, Australia.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), pp. 2042-2043.

The corresponding values of E_0 (Young's modulus) are 151 and 139 lb. per sq. in., giving a mean of 145 lb. per sq. in., or 10.2 kg. per sq. cm. The specimen was also tested as a beam resting on rollers 6 in. apart. The corresponding value of E was determined by the ordinary deflection formula,

$$D = \frac{W\,l^3}{48\,E\,I}$$

Measurements were made as given in Table 1. In the first pair, the side 0.49 in. long was vertical, and in the second pair the side 0.51 in. long was vertical; the specimen was inverted between each pair of measurements, ample time being given to allow the india rubber to recover from the previous deflection. The modulus in bending is called E_1—its mean value is 174 lb. per sq. in., or 12.2 kg. per sq. cm.

TABLE 1.—BENDING OF INDIA RUBBER PRISM.

Load:		Deflection, in inches.	E_1, in pounds per square inch.
In grammes.	In pounds.		
50	0.11	0.55	180
50	0.11	0.60	165
50	0.11	0.51	180
50	0.11	0.54	170

The difference between these results was surprising, but a repetition of the experiment gave similar results, namely, $E_0 = 125$ lb. per sq. in. and $E_1 = 172$ lb. per sq. in.

In case the value of Young's modulus for the cylinder might differ from that in the specimen a separate determination of its value was made. For this purpose the cylinder (with the wooden base removed so that it was open at both ends) was allowed to deform under its own weight by placing it horizontally on a table in four different positions, namely, with each of four quadrants A, B, C, and D, in turn, on top. The mean elongation, H, of the horizontal diameter in each of the four positions was observed and also the mean contraction, V, of the vertical diameter.

It can be shown that:

$$H = \frac{w\,a^4}{E\,I}\left(2 - \frac{\pi}{2}\right) = 0.4292\,\frac{w\,a^4}{E\,I}$$

$$V = \frac{w\,a^4}{E\,I}\left(\frac{\pi^2}{4} - 2\right) = 0.4674\,\frac{w\,a^4}{E\,I}$$

in which

a = mean radius of the cylinder = 7.45 cm.
l = length = 18 cm.
$I = \frac{1}{12}\,l\,t^3 = 3.45$ cm.^{4}
t = thickness = 1.32 cm.
w = weight per unit length of the mean circumference.

Since the total weight, $W = 1\,091$ grammes,

$$w = \frac{W}{2\pi a} = \frac{1\,091}{2\pi \times 7.45} = 23.3 \text{ grammes per cm.}$$

After the deformation the mean horizontal diameter was found to be 15.56 cm. and the mean vertical diameter, 14.19 cm.

Hence,

$$H = 15.56 - 14.90 = 0.66 \text{ cm.}$$

$$V = 14.90 - 14.19 = 0.71 \text{ cm.}$$

Using these values it is found that $E_1 = 13.5$ and 13.7 kg. per sq. cm., respectively. The mean value is $E_1 = 13.6$ kg. per sq. cm.

No satisfactory method of determining E_0 for the circumferential stress suggested itself; therefore the assumption was made that $E_0 = E_1 = 13.6$ kg. per sq. cm.

Description of Experiments

In making an experiment a set of measurements was taken first with the cylinder empty and then filled with mercury.

Two diameters, $A\,C$ and $B\,D$, at right angles, were measured at heights, 0, 1, 2, 3, * * * 6 in. A, B, C, D thus divide the cylinder into four quadrants. No record was taken of the temperatures. The temperature in the shade recorded at the Weather Bureau (near the University of Melbourne) on the date of Experiment No. 1 (October 13, 1924) was 76° Fahr.; on the date of Experiment No. 2 (November 11, 1924), it was 65° Fahr.; and on the date of the repetition of Experiment No. 2 (December 12, 1925), 69° Fahr.

The calipers used (both inside and outside types) were fitted with clamping screw and slow-motion attachment. The measurements were estimated to 0.01 cm., but the error of any reading possibly amounted to 0.03 cm. on account of the difficulty of ensuring that the calipers did not press into the rubber.

In making the outside measurements, one observer would hold a leg of the calipers in the position, say, A_3 (Position A, 3 in. above the base), while the other observer set the other leg so as to be just in contact at C_3 (Position C, 3 in. above the base); the spread of the legs, in centimeters, was then read.

The results of the measurements for the base simply supported (taken on October 13, 1924) are shown in Fig. 1 (*a*). For the case of the base encastrée the results of the measurements (taken on November 12, 1924) are shown in Fig. 1 (*b*). In the latter experiment there was an abrupt change of direction in the surface at the base when the cylinder was full of mercury; as nearly as could be judged by the eye (using a straight-edge) it was straight from 0 to 1 in. above the base.

The abrupt change of direction was due to the considerable local stresses at the junction of the binding wire and the free surface of the india rubber. The same phenomenon can be observed by placing a piece of india rubber ½ in. square and 3 or 4 in. long in a vise and bending it over by one finger held about 1 in. from the jaws. The side away from the finger remains practically straight and shows an abrupt angle at the vise while the side on which the finger is pressed is curved.

A repetition of this experiment was made on December 12, 1925, the internal diameter, $A\ C$, being measured at each $\frac{1}{2}$ in. of height from 0 to 1 in. These measurements are also recorded in Fig. 1 (b). The displacements are taken as one-half the difference in diameter at the base and at the given height.

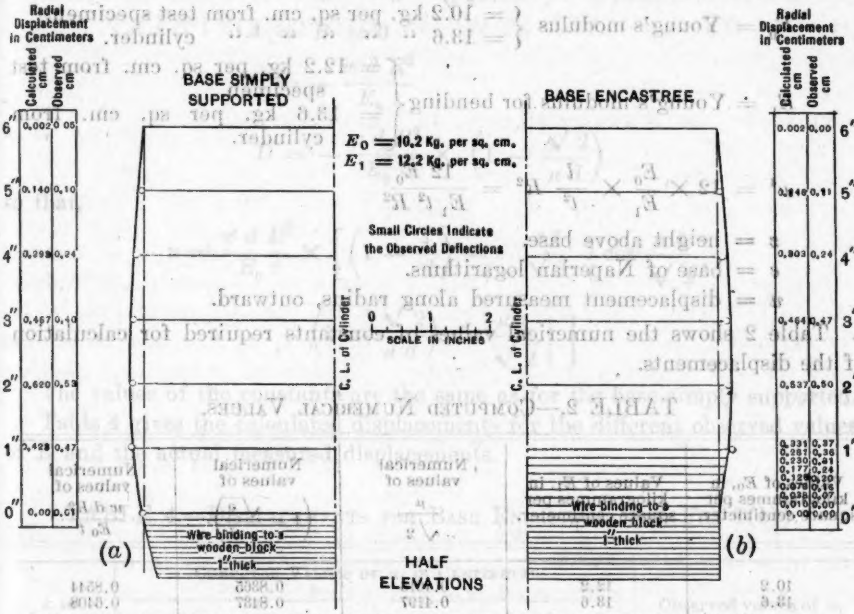

FIG. 1.—EXPERIMENTS ON DEFLECTIONS OF AN INDIA RUBBER CYLINDER FILLED WITH MERCURY.

CALCULATION OF DISPLACEMENTS

1.—Base Simply Supported.—The pressure at the depth, z, is,

$$p = w\ (d - z) = w\ d \left(1 - \frac{z}{d}\right)$$

The limiting conditions are: (1) when $z = 0$, $u = M = 0$, that is, $u = \frac{d^2\ u}{d\ z^2} = 0$, and (2) when $z = d$, $M = F = 0$, that is, $\frac{d^2\ u}{d\ z^2} = \frac{d^3\ u}{d\ z^3} = 0$.

Using these conditions in Equations (16) and (17) of the writer's paper on "Arched Dams",* it is found that, neglecting terms involving $e^{-\frac{\mu\ d}{\sqrt{2}}}$, the constants,

$$A = B = D = 0$$

and the constant,

$$C = -\frac{w\ d\ R^2}{E_0\ t}$$

and, therefore,

$$u = \frac{w\ d\ R^2}{E_0\ t}\left(\left(1 - \frac{z}{d}\right) - e^{-\frac{\mu\ z}{\sqrt{2}}} \cos \frac{\mu\ z}{\sqrt{2}}\right)$$

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 2042–2043.

in which,

w = weight of 1 cu. cm. of mercury = 13.6 grammes per cu. cm.

d = depth, in centimeters = 6×2.54 = 15.24 cm.

R = mean radius = 7.45 cm.

t = thickness = 1.32 cm.

E_0 = Young's modulus $\begin{cases} = 10.2 \text{ kg. per sq. cm. from test specimen.} \\ = 13.6 \text{ " " " " " cylinder.} \end{cases}$

E_1 = Young's modulus for bending $\begin{cases} = 12.2 \text{ kg. per sq. cm. from test specimen.} \\ = 13.6 \text{ kg. per sq. cm. from cylinder.} \end{cases}$

$$\mu^4 = 12 \times \frac{E_0}{E_1} \times \frac{l}{t^2} R^2 = \frac{12\, E_0}{E_1\, t^2\, R^2}$$

z = height above base.

e = base of Naperian logarithms.

u = displacement measured along radius, outward.

Table 2 shows the numerical values of constants required for calculation of the displacements.

TABLE 2.—COMPUTED NUMERICAL VALUES.

Values of E_0, in kilogrammes per square centimeter.	Values of E_1, in kilogrammes per square centimeter.	Numerical values of $\frac{\mu}{\sqrt{2}}$.	Numerical values of $\left(1 - \frac{\sqrt{2}}{\mu\, d}\right)$.	Numerical values of $\frac{w\, d\, R^2}{E_0\, t}$.
10.2	12.2	0.4018	0.8365	0.8544
13.6	13.6	0.4197	0.8437	0.6408

Table 3 gives the displacement at each inch corresponding to the values of E_0 and E_1 and, for comparison, the observed displacements from Fig. 1 (*a*).

TABLE 3.—DISPLACEMENTS FOR BASE SIMPLY SUPPORTED (SEE FIG. 1 (*a*)).

z, in inches.	Computed values of u, in centimeters.		Observed values of u, in centimeters.
	$E_0 = 10.2$, $E_1 = 12.2$	$E_0 = 13.6$, $E_1 = 13.6$	
0	0.000	0.000	— 0.01
1	0.423	0.427	0.47
2	0.620	0.468	0.53
3	0.467	0.347	0.40
4	0.293	0.218	0.24
5	0.140	0.104	0.10
6	0.002	0.001	— 0.05

2.—Base Encastree.—As before, $p = w\, d \left(1 - \frac{z}{d}\right)$. The limiting conditions are:

When $z = 0$,

$$u = \frac{d\, u}{d\, z} = 0$$

and when $z = d$,

$$\frac{d^2 u}{d\,z^2} = \frac{d^3 u}{d\,z^3} = 0$$

from which it follows that, neglecting $e^{-\frac{\mu d}{\sqrt{2}}}$,

$$A = B = 0$$

$$C = -\frac{w\,d\,R^2}{E_0\,t}$$

$$D = -\frac{w\,d\,R^2}{E_0\,t} \times \left(1 - \frac{\sqrt{2}}{\mu\,d}\right)$$

so that,

$$u = \frac{w\,d\,R^2}{E_0\,t} \times \left[\left(1 - \frac{z}{d}\right) - e^{-\frac{\mu z}{\sqrt{2}}} \left\{ \cos\frac{\mu z}{\sqrt{2}} + \left(1 - \frac{\sqrt{2}}{\mu\,d}\right) \sin\frac{\mu z}{\sqrt{2}} \right\}\right]$$

The values of the constants are the same as for the base simply supported. Table 4 gives the calculated displacements for the different observed values of E and the actual measured displacements.

TABLE 4.—DISPLACEMENTS FOR BASE ENCASTRÉE (SEE FIG. 1(b)).

z, in inches.	COMPUTED VALUES OF u, IN CENTIMETERS.		Observed values of u, in centimeters.
	$E_0 = 10.2$ $E_1 = 12.2$	$E_0 = 13.6$ $E_1 = 13.6$	
0	0.000	0.000	0.00
⅛	0.010	0.008	0.00
¼	0.038	0.029	0.07
⅜	0.078	0.058	0.16
½	0.126	0.095	0.20
⅝	0.177	0.134	0.24
¾	0.230	0.172	0.31
⅞	0.281	0.210	0.36
1	0.331	0.265	0.37
2	0.537	0.413	0.50
3	0.464	0.348	0.47
4	0.308	0.224	0.24
	0.146	0.106	0.11
	−0.002	−0.001	0.00

ACKNOWLEDGMENTS

The writer's thanks are due to Professor J. N. Greenwood and the Staff of the Engineering School of Melbourne University for their valuable assistance in the preparation of apparatus and conduct of experiments; also to his Assistant, Mr. C. L. Sanders, who carried out the actual measurements with him.

THE DESIGN, CONSTRUCTION, AND OPERATION OF A SMALL SEWAGE DISPOSAL PLANT

By Franklin Hudson, Jr.,* Jun. Am. Soc. C. E.

Synopsis

This paper describes the design, construction, and operation of a small sewage disposal plant at Stroud, Okla., a town of approximately 2 000 inhabitants. This plant, which was designed, built, and operated for a time, by the writer, under the direction of Webster L. Benham, M. Am. Soc. C. E., has been referred to by the Sanitary Engineer of Oklahoma, as the most complete in the State.

Design

In 1920 Stroud, Okla., had a population of 1 361. This figure, however, had little bearing on the design of the sewage disposal plant as the city was growing rapidly due to the development of a sizable oil field near-by.

After considering the probable maximum future flow, and relying partly on experience in Oklahoma oil towns, it was decided to build a plant capable of disposing of the sewage from a population of 3 000. All units were arranged so that the capacity of the plant could be doubled, in the event that oil development continued.

Surveys showed that sufficient head was available between the invert of the main sewer and the flow-line of the creek to permit the use of almost any gravity type of treatment plant. The type of plant selected was calculated to give the best results with a minimum of attention which in small towns of the Middle West seems to mean no attention whatsoever.

A conventional Imhoff tank, with screen chamber and sludge bed, was designed, together with a dosing tank, sprinkling filter, secondary settling tank with sludge bed, and chlorinator. During construction, a small circular brick incinerator was added, adjacent to the screen chamber, so that screenings could be shoveled directly on the grates for future burning, thus insuring a much more sanitary neighborhood in the vicinity of the screens.

The screen chamber consisted of a rectangular box having a width approximately twice the diameter of the main sewer and a length sufficient to accommodate the iron bar screen and a small operating platform. The screen proper was ½-in. by 1½-in. bars, spaced 1½ in., center to center, on a slope of 3

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horizontal to 1 vertical. The walls of this chamber were built above the top of the Imhoff tank as the small platform had been arranged to allow the entrance of sewage to the plant in case the screen should become completely clogged.

A single, reverse-flow, Imhoff tank (Fig. 1), rectangular in plan, except that the sewage troughs were rounded at the corners, was planned to care for a maximum flow of 125 000 gal. per day, based on an hourly flow equal to one-eighteenth the daily flow.

This flow, with a detention period of 2 hours and a longitudinal velocity of 0.3 ft. per min., was used to determine the dimensions of the sedimentation chamber. The sludge digestion compartment was proportioned for 1.3 cu. ft. per capita per year for a storage period of 1 year, with an additional allowance of 2½ ft. from the calculated maximum sludge level to the slots in the bottom of the sedimentation chamber.

With these data the tank was so dimensioned in plan as to allow 40% of the total area to be used for a gas vent. A slope of 1.8 horizontal to 1 vertical was used in the design of the hopper bottoms of the sludge compartment and the 8-in. sludge pipes leading to the sludge trough discharged under a head of 5 ft., measured from the normal sewage level in the tank.

The two sludge beds, with 2 in. of sand, 2 in. of fine gravel, and 8 in. of coarse gravel composing the surface, were to be of the simplest construction possible having low masonry walls to prevent the surrounding earth from washing in. It was assumed that a 1-ft. stand of sludge would be drawn on each bed ten times each year; this approximated 4 persons per sq. ft. of sludge-bed area, which was thought to be satisfactory.

Calculations showed that a dosing tank of 1 000 gal. capacity, operating under maximum conditions, would have a discharge time of about 4 min. and a filling period of 9 min., making each complete cycle of operation occupy 13 min. The height of the tank was determined from a discharge at high head, which would avoid over-spraying, and one at low head, which would be 40% in excess of the inflow. Siphon and piping losses were estimated by several formulas. For economy, and due to its position at one corner of the sprinkling filter, this tank was designed with vertical walls, which, from later performance, seemed quite satisfactory even if not of the most approved cross-section to give perfect distribution under varying heads.

Filter material of 1 in. to 2½-in. crushed limestone, 6 ft. deep, was chosen, with a maximum rate of filtration of 2 700 000 gal. per acre per day, as the basis for planning the sprinkling filter (Fig. 2). Spray nozzles, ⅞ in. in diameter, spaced 10 ft., gave a proper distribution of sewage. Each one of the 4-in. cast-iron pipe laterals leading from the 10-in. main served four nozzles.

Provision was made for by-passing sewage from the dosing tank direct to the secondary settling tank so that all nozzles could be cleaned and repaired. To provide proper ventilation, triangular wells were planned at each corner of the filter to which the ventilation ducts in the under-drain system were connected. These ducts were formed by omitting one section of the split, notched, under-drain tile in each line and covering the resulting opening with

horizontal to 1 vertical. The walls of this chamber were built above the top of the Imhoff tank as the small platform had been arranged to allow the entrance of sewage to the plant in case the screen should become completely clogged.

A single, reverse-flow, Imhoff tank (Fig. 1), rectangular in plan, except that the sewage tr[illegible] at the corners, [illegible] to care for a maximum flow of [illegible]000 gal. per day, based on an hourly flow equal to one-eighteenth the daily flow.

This flow, with a detention period of 2 hours and a horizontal velocity of 0.3 ft. per min., was used to determine the dimensions of the sedimentation chamber. The sludge digestion compartment was proportioned for 1.3 cu. ft. per capita per year for a storage period of [illegible] with an additional allowance of 2½ ft. [illegible] the calculated maximum sludge level [illegible] bottom of the sedimentation chamber.

With these main tank [illegible] of the total area [illegible] used for the gas vent. [illegible] to 1 vertical was used in the design of the hopper bottoms of the sludge compartment and the 8-in. sludge pipes leading to the sludge trough discharged under a head of [illegible] ft., measured from the normal sewage level in the tank.

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Calculations showed that a dosing tank of 4 000 gal. capacity, operating under maximum conditions, would have a discharge period of about [illegible] min. and a filling period of 9 min., making the complete cycle of operation about 12 min. The height of the tank was determined from a discharge at high head which would [illegible] overlying, and one at low head, which would be 10% in excess of the inflow. [illegible] several formulas, [illegible] speaking [illegible], this tank was designed with vertical walls which, in later performance, seemed quite satisfactory even if [illegible] cross-section to give perfect distribution under varying heads.

Filter material, 1 in. to ¾-in. crushed limestone, 6 ft. deep, was chosen, with a maximum rate of filtration of 2 700 000 gal. per acre per day, as the basis of design for the sprinkling filter (Fig. [illegible]). [illegible] nozzles, [illegible] in. in diameter, spaced 13 ft., gave a proper distribution of sewage. Each [illegible] from the laterals leading from the main [illegible] feed four nozzles.

Provision was made for by-passing sewage from the [illegible] direct to the secondary settling tank so that all nozzles could be cleaned and repaired. To provide proper ventilation, triangular wells were planned at each corner of the filter to which the ventilation ducts, in the under-drain system were connected. These ducts were formed by omitting one section of the split, notched, under-drain tile in each line and covering the resulting opening with

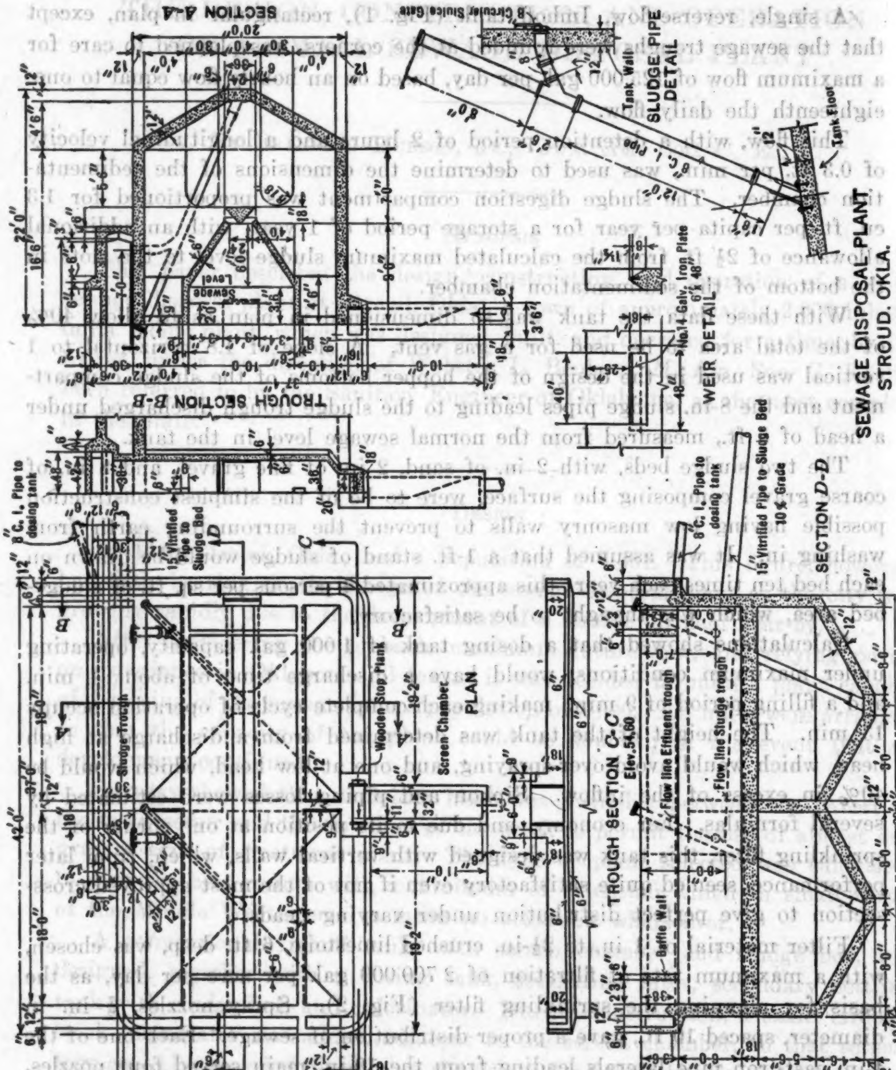

FIG. 1.

precast concrete slabs. The main under-drain was vented at its upper end by the use of 12-in., vitrified clay pipe with a perforated plug.

It was found that a satisfactory arrangement could be devised whereby the secondary settling tank and sludge bed, and the chlorinator (Fig. 3), could be built as one structure. The secondary settling tank was designed for ½-hour detention with 20% of the basin capacity reserved for settled sludge. No elaborate method of sludge control was proposed, other than to skim the liquid into the chlorinator mixing chamber at a point as remote as possible from the inlet. Slopes of 1 horizontal to 1 vertical composed the floor of the tank.

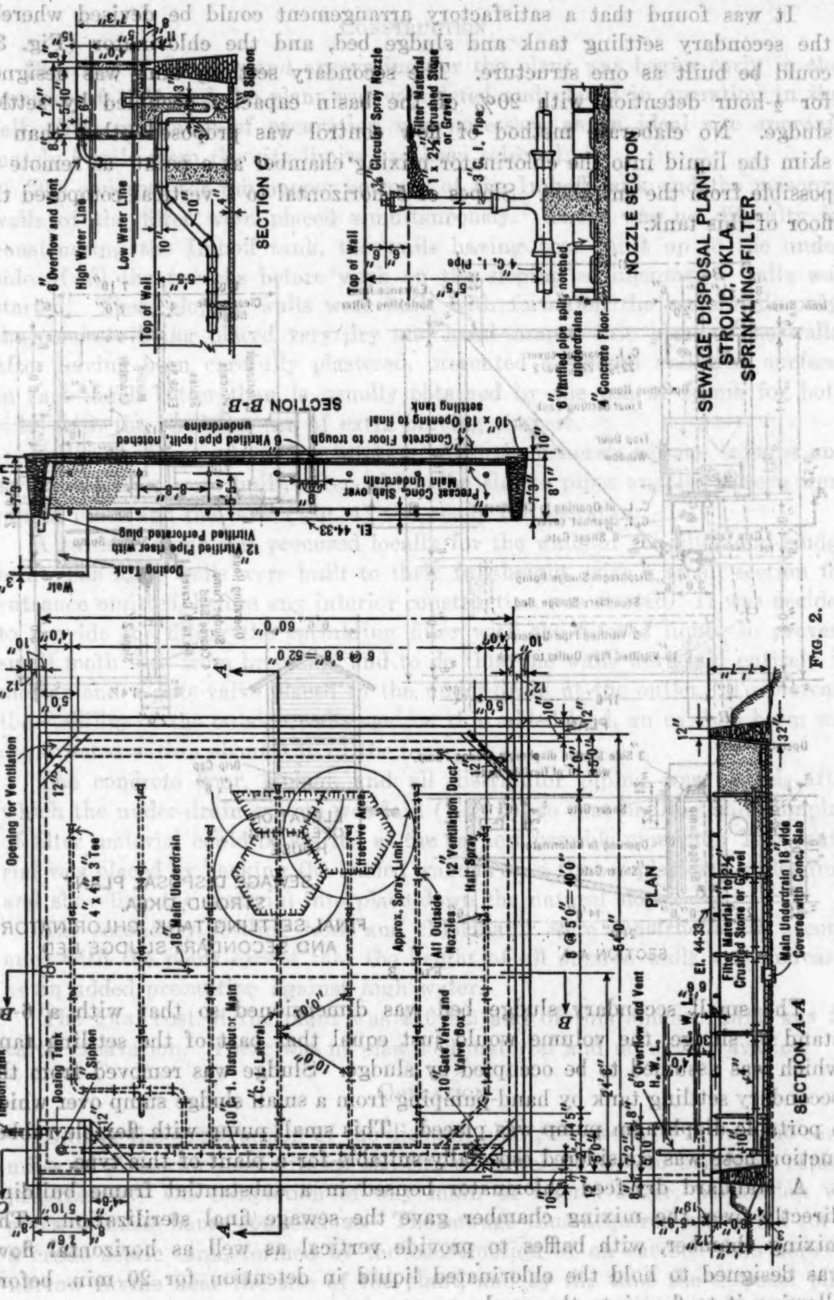

FIG. 2.

The small secondary sludge bed was dimensioned so that with a 6-in. stand of sludge, the volume would just equal that part of the settling tank which was assumed to be occupied by sludge. Sludge was removed from the secondary settling tank by hand-pumping from a small sludge sump over which a portable diaphragm pump was placed. This small pump, with flexible suction hose, was considered quite suitable for a plant of this size.

A standard dry-feed chlorinator, housed in a substantial frame building directly over the mixing chamber gave the sewage final sterilization. The mixing chamber, with baffles to provide vertical as well as horizontal flow, was designed to hold the chlorinated liquid in detention for 20 min. before allowing it to flow into the creek.

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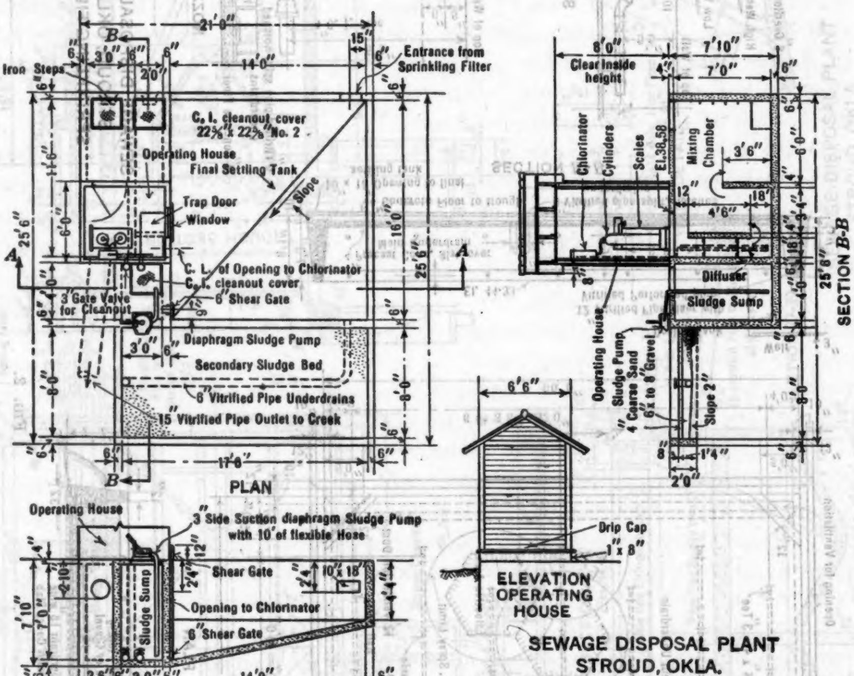

SEWAGE DISPOSAL PLANT
STROUD, OKLA.
FINAL SETTLING TANK, CHLORINATOR AND SECONDARY SLUDGE BED

FIG. 3.

The small secondary sludge bed was dimensioned so that with a 6-in. stand of sludge, the volume would just equal that part of the settling tank which was assumed to be occupied by sludge. Sludge was removed from the secondary settling tank by hand-pumping from a small sludge sump over which a portable diaphragm pump was placed. This small pump with flexible rubber suction hose was considered especially suitable for a plant of this type.

A standard dry-feed chlorinator housed in a substantial frame building directly over the mixing chamber gave the sewage final sterilization. The mixing chamber, with baffles to provide vertical as well as horizontal flow, was designed to hold the chlorinated liquid in detention for 20 min. before allowing it to flow into the creek.

The foregoing data are those on which the design of the plant was based and all plans were prepared.

Construction

Clearing of the site and excavating for the plant was begun early in the summer of 1924 and the plant was completed and placed in operation in the fall. The minimum of excavation was necessary as an ideal site approximately ¾ mile from the city limits had been selected.

The concrete for the hopper bottoms of the Imhoff tank and the masonry walls of the filter were placed simultaneously. There was no difficulty in constructing the Imhoff tank, the walls having been built up to the under side of all the troughs before work on the sloping sedimentation walls was started. These sloping walls were built with forms on the under side only, the concrete being placed very dry and hand-tamped into place. The walls, after having been carefully plastered, presented a smooth and true surface, in fact much better than is usually obtained by the use of forms for both sides with the resulting use of extra bracing timbers.

Following the work on the interior walls, the concrete sewage troughs and screen chamber were built, after which the sludge pipes and bar screen were put in place and the tank (Fig. 4) was ready for operation.

A suitable stone was procured locally for the walls of the filter and sludge bed. The filter walls were built to their full height, with a small section for entrance omitted, before any interior construction was started. It was decided to provide for filling the sprinkling filter with the filtered liquid to prevent small moth flies from breeding, and to do this, the walls were laid entirely in mortar and a gate-valve placed in the under-drain at the outlet. To increase the stability of the outside walls against this added load, an earthen berm was constructed within about 3 ft. of the top.

The concrete floor, siphon, and all distributor piping was placed, after which the under-drain system was laid (Fig. 5), so that hauling and dumping of filter material could be started at the earliest possible moment. This material was placed by backing the loaded wagons on a timber platform, unloading, and shoveling the material into place down the natural slope.

The secondary settling tank and chlorinator were constructed in accordance with the plans except that the height of all outside walls was increased as an added precaution against high water.

The total cost of the plant was $16 872 and of this amount $611 was for earth excavation. There was no classified material and no wet excavation.

Operation

During the construction of the disposal plant, the work of laying the seven miles of pipe which constituted the Stroud sewerage system had been rapidly progressing. On completion of any main or lateral sewer, permission was given to make house connections. The sewage thus acquired was cared for in a crude septic tank, formed by the construction of an earthen dam across a narrow ravine near the site of the plant, and by the time the disposal plant was ready for operation a considerable flow of sewage was available.

Normal operation began after filling the Imhoff tank, and all units (Fig. 6) functioned satisfactorily. A simple set of rules was developed to enable the plant to be operated by devoting only a part of one day each week to the task.

It was found necessary once a month to remove all nozzles from the risers and allow the dosing tank to discharge once or twice so as to remove the accumulated débris. The small incinerator was used each month and about all that was required each week was to clean the screens, check the chlorine dosage, and regulate the Imhoff tank.

Young boys were particularly troublesome before the fence around the plant was finished, and even afterward they seemed to have little trouble in getting over the "non-climbable" barbed wire. The chlorinator building was a fair target for young hunters until lurid "danger" signs were painted on each wall. Even the nozzles had an attraction, as it was discovered one morning that all of them had been broken off. After they had been replaced, however, no further damage occurred.

Excellent results were obtained during the eight months the plant was operated under the direction of the writer. Many stability tests were made, indicating a uniformly good effluent. Monthly operating costs during this period were about as follows:

Chlorine	$7.00
Plant maintenance....................	3.50
Sludge removal.......................	1.50
Total........................	$12.00

An inspection of this plant was made after more than a year of operation by the city, and it was found to be in a condition similar to that of other plants in small towns and for which there seems to be no remedy. The screen was completely clogged, the Imhoff tank partly covered with scum, and all the nozzles on the filter bed had been removed to save cleaning them. At the secondary settling tank, the sludge was noticeable on the surface and no chlorine was being used for sterilization. On inquiry it was found that the city workmen had made only three or four trips to the plant after the writer had left. After a general cleaning up, replacing of nozzles and new chlorine cylinder, the plant once again functioned properly.

Conclusion

From the knowledge gained by operating this sewage disposal plant, the writer suggests the following changes that would tend to increase the practical efficiency.

Imhoff Tank.—The substitution of slots, on the flow line of the sewage troughs at the entrance to the sedimentation chamber, in place of the weirs as constructed, which would eliminate considerable trash that collects and settles back of the weirs, causing reversal of flow to be slightly unsatisfactory; and the addition of a 4-mesh screen over the entrance to the dosing tank, with a suitable emergency overflow, to prevent sizable floating matter from clogging the nozzles.

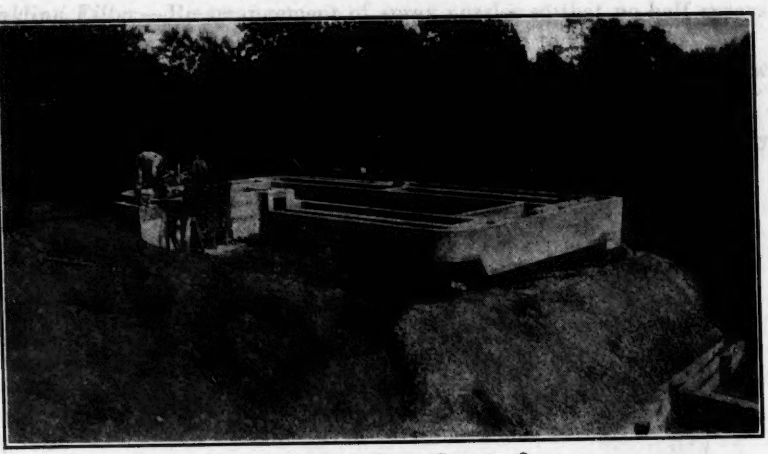

FIG. 4.—IMHOFF TANK, STROUD, OKLA.

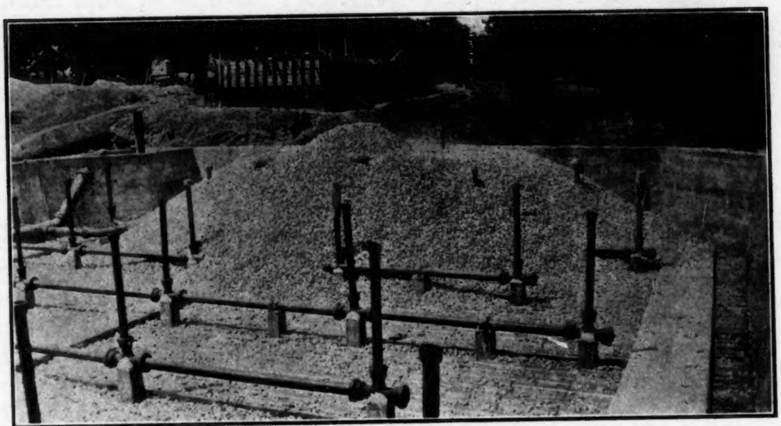

FIG. 5.—CONSTRUCTION OF SPRINKLING FILTER, STROUD, OKLA.

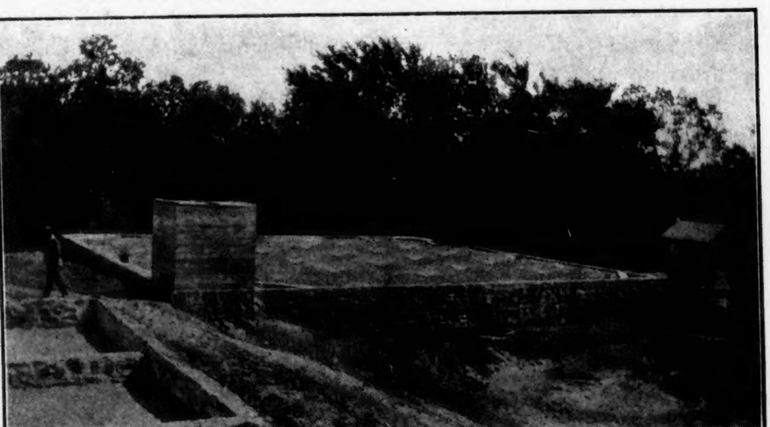

FIG. 6.—DOSING TANK AND SPRINKLING FILTER, STROUD, OKLA.

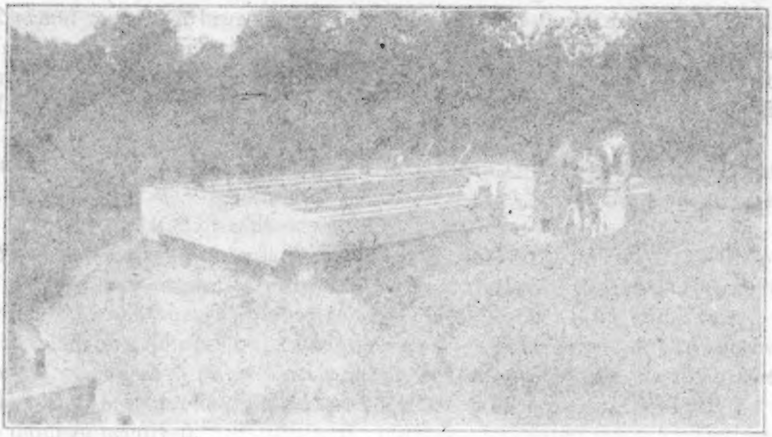

Fig. 4.—Imhoff Tank, Stroud, Okla.

Fig. 5.—Construction of Sprinkling Filter, Stroud, Okla.

Fig. 6.—Dosing Tank and Sprinkling Filter, Stroud, Okla.

Sprinkling Filter.—Re-arrangement of spray nozzles so that no half sprays would be necessary, as these clogged badly and were hard to maintain.

Secondary Settling Tank.—The addition of vertical baffles to regulate flow so that a more uniform deposition of sludge would result.

These changes have been made in all the more recent plants designed by the writer and they have been entirely satisfactory.

TOWN PLANNING AND ITS RELATIONS TO THE PROFESSIONS INVOLVED*

By John Nolen,† Esq.

There is much in the speaker's mind, and in the minds of most other students of town planning, that is not yet altogether clear. The art and science of town planning still contain many unknown elements. One thing, however, is fairly clear, namely, that town planning, if it is to accomplish its purposes in a worthy manner, must be broad in the professions it employs. If permanent results of a high order are to follow, the engineer, the architect, the landscape architect, and also the lawyer, and those of other allied professions, must be united in a joint enterprise, one which should have variety, harmony, and unity. The planner of the town himself may be an engineer, an architect, or a landscape architect. Professionally he ought to be technically educated, well trained, and widely experienced in at least one of these three fields, but he must know something vital of them all. Successful town planning cannot be the work of a narrow specialist of a single profession. It calls for versatility. It is engineering plus something; architecture plus something; or landscape architecture plus something; and that "plus" is as indispensable as the direct professional equipment in the more usual and better recognized fields. Moreover, the town planner needs to know both design and construction in those public works and improvements that draw upon and employ the skill of several of these professions jointly, and especially those works that have constantly the double requirement, if it can be so called, of use and beauty. As some one has said, harmony and beauty must be created on a foundation of the practical. One thing more—the town planner must have the social welfare point of view, for the motive back of all town planning is to improve daily living and working conditions. In this field more than in many others, personality plays a large part.

The general town plan, or "master plan", as it is sometimes styled, is not enough. There must be site or local planning. The details of a town development, of all its various features, must be properly done in both plan and elevation. The third dimension enters vitally into any real town development. The general town plan must have its own merit, its own justification; but as the plan of a building is more fundamental than its elevations, so the town plan itself is more important than any or all of its structures. Nevertheless, convenience, order, beauty, and appropriateness are finally realized only in these structures of the engineers, the architects, and the landscape architects.

Note.—Written discussion on this paper will be closed in **January, 1927.** When finally closed the paper, with discussion in full, will be published in *Transactions*.

* Presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926.

† Pres., National Conference on City Planning, Cambridge, Mass.

Mark this point of supreme importance, however, that unless this principle of unity and fitness is followed, these various works of detail may be satisfactory in themselves separately, but as an example of the art of town planning, or as a complete fulfillment of their practical purposes, they will fail. Each separate part must be tested for merit both by itself and by its contribution to the whole.

To understand this subject, some of the chief characteristics of town planning and its point of view must be enumerated, and the nature of engineering, architecture, and landscape architecture indicated briefly, especially as they are employed in community development. Mariemont, Ohio, is a good example of a community developed under the joint effort of men representing various professions. It illustrates problems in design, construction, and organization.

1.—To begin, town planning is a comprehensive, inclusive, synthetic art. Its success depends on having a broad view with a keen sense of proportion, of fitness, and of social values. That is the keynote of the town-planning movement.

2.—Town planning is broad from the point of view of the territory included in its schemes. It embraces wide areas. Dealing only with the parts of a town or with local sections or neighborhoods it cannot work successfully. More and more with the extending radius of modern life, it is becoming regional in character. If well done, town planning design takes its cue from this broad regional viewpoint in which there is a skillful arrangement of each part of a wide territory, assigning it to its most appropriate use and development.

3.—Town planning is comprehensive in embracing all the physical elements of a community. It includes the location, character, and appearance of thoroughfares, railroads, parks and playfields, schools, public and semi-public buildings, street structures, and the sanitary disposal of wastes, etc.

4.—Town planning is broad planning from the viewpoint of time. It is historical. It looks forward, it looks backward. To plan for to-day and to-day alone, or to plan for to-day without regard to yesterday, is not town planning in its full sense. The relationship runs backward and forward, and towns have a long, long life. They endure for centuries. The human limitation of three-score years and ten has no application.

5.—Broad also is the viewpoint of town planning in that it embraces all sides of man's life—animal, social, intellectual, and spiritual. The most commonplace needs of man as an animal must be properly provided for by town planning; indeed these needs—food, shelter, and a place to work—must be considered before anything else. Man, however, is not merely an animal. Man is a spirit. He aims not only to live, but to live well, with increasing freedom and happiness. That means, if it means anything, planning a town and an environment not only for labor but for leisure; and leisure at its best is not idleness, but a different and a higher form of occupation, the facilities for which must be provided largely by the public. Some of the more worthy and satisfying objects of this steadily increasing leisure, combined as it is with the increase of wealth, are to be found in art and science, in play and sport, in the contemplation of the universe, in the enjoyment of nature, in friend-

ship and social life. One can readily see the close relation of these objects, if they are to be shared by all, to a well-planned community.

6.—Another characteristic of the broad nature of town planning is illustrated in its economic, legal, and administrative aspects. Town planning cannot proceed a step without counting the cost. The final conundrum is paying the bills. This often is the check to town planning advance; and yet nothing is more certain than the fact that town planning pays both in direct and indirect ways. Certain features are indispensable for a community, such, for example, as suitable main thoroughfares, land for parks and other public uses, proper provision for railroads, including location of rights of way, passenger and freight stations and railroad yards; also, public building sites, especially well-located land for schools, and areas for housing with adequate and pleasant home grounds and gardens. All these features a town must have, if it is to endure and prosper. The only choice is whether they are to be obtained early with the initial planning when land is cheap and conditions controllable, or later when land is expensive and conditions are fixed and more or less unchangeable. Moreover, a town has only a choice usually of the form of its expenditures. If it does not provide these essential features for town life, the people must pay an equal or greater sum in other and less satisfactory ways. An examination of such subjects, as traffic regulation, recreation, education, housing, showing comparative conditions and comparative costs in various towns would confirm this statement.

7.—Town planning has its legal side and administrative machinery. It involves an understanding of the rights of property compared with the rights of persons; the rights of single individuals compared with the rights of the group. Furthermore, there is the administrative machinery for carrying out the town plan. How to get the thing done is the vital issue. The town plan must be put in action. It must work. Indispensable as is the paper stage, it is the first stage only. The question must be asked, by what means can the plan and its program be carried out?

Broad requirements such as those cited as indispensable for town planning call not only for the usual professional skill, but also for imagination, for the approach that is characteristic of the artist, for one of creative mind, for the point of view that shows power to grasp, to simplify, to express social ideals in terms of a better ordered, less wasteful, more satisfying, and more beautiful town. Mr. Raymond Unwin, the distinguished English town planner, states this particularly well in an address,* which might have the title "The Artist and the Practical Man", in which he says:

"Hitherto the work of town planning has suffered for want of clearer understanding, even on the part of those well versed in the subject, of the difference of faculties and methods needed for success. If the practical man has sometimes thought that complete mastery of the science of the subject would suffice to enable him to practise that which is as much an art as a science, it must be admitted that the artist has at times also imagined that his training and his art have forthwith qualified him to become a planner of towns, forgetting that this particular art is based on an extensive science, which

* "The Architect and His City", printed by permission of the Royal Inst. of British Architects, in *Journal* of The Town Planning Inst., December, 1925, p. 36.

must be at least understood. * * * What the artist specially needs is a sympathetic insight into all the relationships of city life, a realization of the reactions which take place between the city environment and the human society which it clothes and expresses. He needs, in fact, that particular range of knowledge which will enable his imagination to picture the city as it might be, to see the life of the people going forward in it, to see all the different parts and functions in their true relation. He needs this that he may be able to study his vision effectively and mould it to meet the realized conditions, or modify it to avoid the apprehended difficulties. The kind of knowledge needed is extensive rather than intensive; for there must be maintained a degree of detachment from the details of the problem if the city and the life of the city are to be seen fairly and seen whole. The town designer must prepare his imagination for this work by watching and thinking over the phases of city life; meditating on their comparative manifestations in many towns; entering sympathetically into the needs and limitations, musing all the time on visions of how work might be made more efficient and town life more pleasant."

With this statement of the nature of town planning, the part played by the various professions usually employed in carrying out the development work of a community may be considered, as well as the requirements placed on the engineer, the architect, and the landscape architect, and their relation to the town planning authority.

First of all is the engineer. He naturally is the man to come on the ground at the very beginning. The basic feature of his work is the topographical survey. Although it has often been thought so, the topographical survey is by no means the whole of the survey of local conditions. It is, however, the most indispensable element. A recent writer has paid tribute to the part played by the surveyor in the development of the Nation, pointing out that the list of surveyors includes such widely different and gifted men as Washington, Daniel Boone, Andrew Jackson, Thoreau, and Lincoln. A surveyor blazes a trail, he defines boundaries, he must be undismayed by obstacles and barriers. Partly because of the fact that the engineer in the form of the surveyor is first on the ground, much town planning has heretofore been done by the engineer. Usually, however, until recently the engineer has brought to town planning only engineering knowledge and not town planning knowledge, and it has frequently been specialized engineering at that, rather than fundamental engineering. For with the great development that has come in engineering science, the field of engineering has been divided and subdivided and specialized, so that to-day the type of engineer that formerly was so all-embracing, so well known, and still so well respected, termed the Civil Engineer, has become in practice largely a thing of the past.

In a valuable address on "The Engineer and the Town Plan", by James Ewing, Vice-President of the Town Planning Institute of Canada, delivered recently before the City Planning Division of the Society,* Mr. Ewing shows how mere engineering, even when most skillful, often fails in town development unless somehow it can be guided and controlled by town planning knowledge. He illustrates his statements by an analysis of what too frequently has been the work of engineers in connection with city streets,

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1343; also, *Journal*, Town Planning Inst. of Canada, October, 1925, p. 8.

bridges, railroads, etc., and points out that these public works are too often monuments of engineering enterprise and skill, and at the same time lasting memorials of wrong judgment and radically bad planning from the standpoint of a community or of National development. He goes further—he shows convincingly how the natural resources of Canada have been wasted and the Nation burdened with debt because the engineering was so often done from a narrow engineering viewpoint, in place of a broad planning point of view.

As the trained and experienced engineer should, of course, be entrusted with the works of engineering in building a town, so should the trained and experienced architect be entrusted with works of architecture. Community development, however, involves something more than the construction or mere assembling of engineering, architectural, and landscape architectural works, meritorious as these may be. It involves the regulation and control of all these constructions in the interest of the whole, for only in this way can the maximum of convenience, good construction, and right appearance be secured.

The architect's work as architect, requires provision for stability, convenience, and beauty, also a nice regard for cost; but a building, be it large or small, might meet all these provisions and yet fail when tested as to its place in community development, because of a lack of proper relation to other architectural works, to engineering, or to landscape architecture, or to some element or feature of the town plan itself. Therefore, the architectural works of a town should be done, if possible, by an architect who himself understands town planning, or else his work should be directed and approved from the town planning point of view, represented by a qualified individual or group of individuals possessed of a variety of town planning knowledge and experience. For a happy fusing of all the complex works of the various professions is the desired goal. Co-ordination is essential.

There is little to be gained by attempting to determine whether the engineer, the architect, or the landscape architect is better fitted for the work of town planning. Each has his own peculiar advantages and limitations. In the United States, the landscape architect has taken a prominent part in town planning for two reasons: First, for several generations the profession has had the good fortune of having a leading firm of gifted men, with high ideals of professional work and public service and a developed social sense, who, in turn, have trained many younger men. This firm has put into landscape design more of the spirit and requirements of town planning than have yet manifested themselves in any other direction. It is true the landscape architect has a natural call toward town planning. He thinks always of buildings in relation to site and topography. He is concerned with their approaches, their background, and their environment. He has regard for natural features of beauty, and their value for recreation or for other important town uses. He is trained in broad design and composition, and the arrangement in convenient, orderly, and agreeable fashion of structures often primarily the work of engineers or architects. The landscape architect thus turns naturally from gardens to parks, from parks to civic centers and streets, from streets to suburban development and land subdivision, and from land

subdivision to town planning. There is an almost inevitable evolution, step by step, not only in the professional technique of planning, but also in weighing the claims of all the various elements of convenient arrangement, requirements of construction, cost, and appearance, so that the result may be as nearly right as it can be, from many points of view.

A second reason for the high place occupied by the landscape architect in town planning, a reason that is at once both an effect and a cause, is that the most complete course of specialized training in town planning offered by an American university is most closely related to the advanced technical courses and research work in landscape architecture. These courses of instruction, however, are equally open to engineers and architects, and have been taken by many men already well trained in these professions. One of the most valuable discussions of this phase of the subject is contained in the series of articles by Mr. Thomas Adams.* The gist of the matter is well expressed by Mr. Adams in these words:

"The appointment of a special visiting committee of the School of Landscape Architecture at Harvard may lead to interesting developments in connection with the teaching of town planning, which in Massachusetts includes city planning. The appointment of the committee should promote further co-operation between the schools of architecture and landscape architecture rather than tend to create any line of cleavage between them. That is agreed to be highly desirable. Both the architect and the landscape architect who intend to specialize in town planning need a higher degree of training in the principles and methods of civic design and more understanding of the reciprocal relations of all the factors in city and town development. The landscape architect *per se* is no more a town planner than the architect, or the architect than the engineer—and no member of any of the three professions has a claim to be a town planner in a more comprehensive sense than the other.

"A member of each profession has the knowledge and qualifications needed as a foundation for making the town planner, but specialized training is needed to be superimposed upon that foundation to make either the architect town planner, the landscape architect town planner, or the engineer town planner. The relative degree of importance of either in town planning will be a matter of personality. Until recently the field of modern town planning in America has appeared to be the specialty of the landscape architect, and no one will question the fact that some distinguished landscape men have given reality to any special claims their group may have appeared to have for the cultivation of that field. Moreover, in Harvard and elsewhere, special teaching in phases of town planning to landscape students has given further stability to these claims. Similar teaching has not been given, or at any rate, to anything like the same extent, to architects and engineers."

Town planning, as far as the special subject of this paper is concerned, finally simmers down very largely to the relation between design and construction, and the methods by which design can be translated successfully into construction, keeping always in view the element of cost and the legal authority with which to proceed. There must be wider training in design of those who have to do primarily with construction, and a larger experience in construction of those who have to do primarily with design. The town plan itself must show an orderly distribution of all its parts, with a nice regard

* *Journal*, Am. Inst. of Architects, Vol. 10, 1922, p. 101.

for the practical requirements of each part, and its esthetic potentialities; and there must be foresight and vision as to what is involved in the social and civic life of the community in the execution of the plan. Detail planning must follow general planning, and again the town planning point of view must prevail. Finally, actual construction takes its place upon the ground in the public utilities, or raises its head above the ground in architectural or engineering structures. Here, once more, the local purposes must be served, but with these local purposes, if there is knowledge, skill, good sense, and co-operation, the spirit of town planning enterprises may be successfully invoked to give, under all the circumstances, the best possible results. In human affairs which have the variety and complexity of town planning, perfection is not possible, but a much higher degree of public convenience and private enjoyment may be secured with a considerable saving of costs, and a distinctly better quality of urban beauty obtained, if a sound method of organization is used in harmonizing the work of the various professions with town planning principles and ideals.

THE DEVELOPMENT OF MARIEMONT, OHIO*

By Frederic H. Fay,† M. Am. Soc. C. E.

Mariemont, Ohio, a town in the making, situated just outside the city limits of Cincinnati, is a practical example of the application of the principles of community development laid down by Mr. Nolen‡ and is the result of the joint effort of men representing various professions. At Mariemont, which is patterned after the English garden cities, American methods, combined with modern city planning ideas, good architecture, and sound engineering practices, are creating a community which is unique in the history of town planning in the United States.

Most municipalities are founded by men; Mariemont's founder is a woman, Mrs. Mary M. Emery, one of Cincinnati's best loved citizens. It is the result of many years of planning by herself and her associates, and is only one of the numerous ways in which she is making her wealth of real and lasting service. The new town of Mariemont, however, is not a philanthropy nor in any way paternalistic; it is not merely an undertaking to build more houses; it is a real estate development on normal American lines, except that profits are restricted to a low figure.

Purpose

Mariemont is intended primarily as a place of residence for families of widely different economic standing, and especially for wage earners. It does not provide homes for the very poor; experience thus far has shown the practical impossibility of providing new homes for people of the lowest economic scale. Mariemont is an outstanding example of the fact that, with careful planning, and with the savings in cost resulting from large-scale construction, it is possible to provide practical, convenient, and well-made homes, amid ideal surroundings for people of moderate means. It is not a laboratory for sociological experiments in the problem of housing. It does not follow the English plan of co-partnership building and ownership. People in America are still too individualistic in their attitude and action to take readily to co-operative housing schemes.

A National Exemplar

At Mariemont, individual home builders, and projectors of towns and subdivisions near great cities, will find a practical demonstration, not only of

Note.—Written discussion on this paper will be closed in January, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

* Presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926.

† Cons. Engr. (Fay, Spofford, and Thorndike), Boston, Mass.

‡ See p. 1612.

careful planning on what are believed to be correct city planning principles, but also of the advantages of modern methods in town building and of the value of beauty both in placing and designing a home. In these respects Mariemont may be called "a National exemplar".

Town Site

The site of Mariemont, about nine miles east of the business center of Cincinnati, is a slightly undulating plain, formerly rich farms and woodland, extending from the base of the steep slopes of Indian Hill, which protects it from the sweep of north winds, to the edge of a wooded bluff overlooking the Little Miami River and its broad level valley 100 ft. below. Across the valley may be seen other Ohio hills and in the far distance the rugged Kentucky hills on the southerly bank of the Ohio River. As the prevailing winds in this locality are from the southwest, there are always, in summer, cooling breezes up the Little Miami Valley over the Mariemont plain. In its landscape setting and other natural features the site is most attractive as a place of residence.

General Description

The general plan of development of Mariemont, covering a tract of about 420 acres, provides for a complete town to include homes for 6 000 to 7 000 people (Fig. 1). Future development of land bordering this tract will probably result in Mariemont becoming ultimately the nucleus of a community of more than double this population. In planning the town, therefore, provision has necessarily been made for meeting the needs of the larger community.

The town is essentially residential. For this reason there are to be no industries directly within the village. Two industrial sections with rail and highway connections are provided, however, one at the westerly end of the property and the other along the banks of the river below the bluff. These are situated so that, although they lie within easy distance of the residential part of the town, there are natural barriers which make them wholly separate and distinct.

A complete town means that all public and business buildings, all parks, playgrounds, and recreational features, and all utilities necessary and desirable for a fully developed municipality are being provided at the outset, together with the building of residences. When construction operations now underway are completed, the town will be fully equipped, everything in it being new and up to date. In its development, however, care has been taken to preserve trees and other natural objects of beauty so that when finished the town will have the appearance and possess the advantages of a long settled community.

The design of Mariemont provides for a town center with a village green, similar to those found in many English country towns. In and around the town center are grouped the principal buildings, the town hall, library, the community building, post office, hotel, bank, and theatre, together with stores

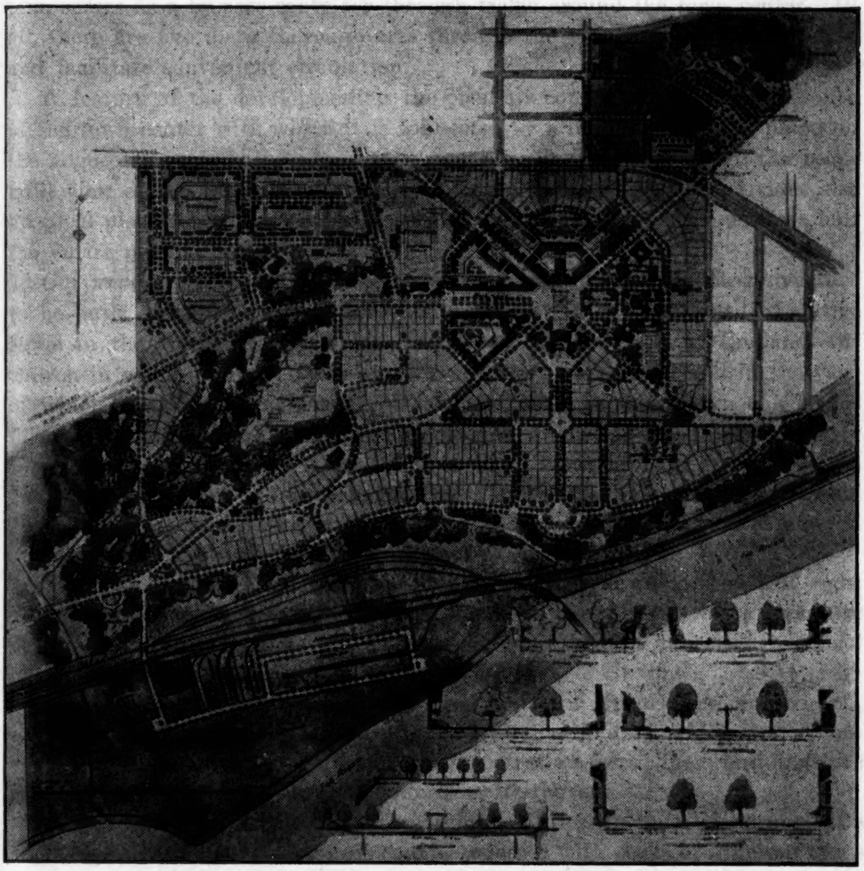

FIG. 1.—LAYOUT OF CENTRAL PORTION OF MARIEMONT, OHIO.

careful planning on what are believed to be correct city planning principles but also of the advantages of modern methods in town building and of the value of beauty both in placing and designing a home. In these respects Mariemont may be called "a National Exemplar."

Town Site

The site of Mariemont, about nine miles east of the historic center of Cincinnati, is a slightly undulating plain, formerly rich farms and woodland [illegible] new and up to date. In its development, however, care has been taken to preserve trees and other natural objects of beauty so that when finished the town will have the appearance and possess the advantages of a long settled community.

The design of Mariemont provides for a town center with a village green similar to those found in many English country towns. In and around the town center are grouped the principal buildings, the town hall, library, the community building, post office, hotel, bank, and theatre, together with stores

and a public market, all so situated as to be within easy walking distance of the residential parts of the town.

The street plan consists of major highways radiating from the town center and secondary residential streets, with service lanes or alleys at the rear of lots. One major thoroughfare from Cincinnati to the east passes through the town center. Along the edge of the bluff, overlooking the river, is a boulevard extending the entire length of the property, a distance of 1½ miles. This boulevard, which will eventually be linked with Cincinnati's Boulevard System, serves as a by-pass route for through traffic around the town center. In all, there are five main thoroughfares through the village to avoid congestion and facilitate convenient circulation.

A feature of the development is the open-air concourse immediately south of the town center with which it is connected by a main avenue. Situated on the edge of the bluff, the concourse preserves as a public privilege the beautiful view of the Little Miami Valley and the distant hills. It provides also an ideal place for public gatherings, being large enough to accommodate half the entire population of the town.

Community Buildings.—The community church (Fig. 2), the first structure to be built, is a memorial to the early settlers. It is constructed of native stone in the Norman style and is a copy of Stokes-Poges, in England, the church in which William Penn and his ancestors worshipped and in the yard of which Gray wrote his famous "Elegy". The huge hand-hewn oak beams forming its rafters were taken from an old mill which had stood in the vicinity for more than a century. The roof is to be of stone brought from England. In its setting among old and rugged trees, and adjoining a small century-old graveyard, this non-sectarian community church possesses a charm and an appearance of age which are noteworthy.

The first of the several projected school buildings, a grade school of eight rooms and a kindergarten (Fig. 3), is already in use. It is a brick structure designed and equipped as an example of best modern practice in school building.

On the slope of Indian Hill overlooking the town Mrs. Emery is providing, as a memorial to her late husband, a hospital adjoining which are a convalescent home and work shops and a demonstration farm; these buildings are grouped in a section of the town known as Resthaven.

Recreation Facilities.—The recreation facilities include playgrounds, a stadium, athletic field, and tennis courts, together with parks of different types and scenic reservations for public use on the bluff above the river. Of the park areas which comprise a total of more than 50 acres, Dogwood Park deserves particular mention. It is in a deep ravine through which flows a small stream where a lagoon is to be constructed. The park site will be preserved in all its natural, rugged beauty.

Housing.—Mariemont is chiefly of interest as a housing development. To meet the needs of those who can afford only low rentals, there are many small flats or apartments, some even of a single room, and also a number of "group-houses", that is, houses attractively grouped together in a row, as shown in

Fig. 4. For residents of more ample means, duplex houses and single houses are being built, as shown in Figs. 5 and 6.

Every residence, from the smallest to the most pretentious, has been skillfully planned and well constructed of durable materials, mostly brick and stone. There are no dark rooms in Mariemont. Every home is provided with running water, bath, and other sanitary facilities, electric light, telephone, and natural gas for cooking. The larger residences, as well as all public buildings, stores, and apartments in and around the town center, are to be heated from a central steam-heating station. Especial care has been taken that no slum district shall ever develop in Mariemont, no matter how thickly populated it becomes. The ample yards in the rear of all habitations, the building setbacks, and the many parks, public squares, and allotment gardens, guarantee perpetual pure air and sunshine. The lots are large and the space between buildings is fixed. Except for a certain few buildings, principally stores and apartments around the town center, no building more than two stories in height is to be erected in Mariemont.

Architectural Work

The architectural planning of Mariemont has been apportioned among twenty-six architects of high standing in Cincinnati, Boston, Mass., New York, N. Y., and Philadelphia, Pa. The result is a pleasing variety and treatment of architectural design, both of residential structures and of community buildings. There is complete absence of the monotony which frequently characterizes large-scale community development. The architectural work has been co-ordinated so as to obtain attractive results in accordance with the carefully thought out plan of the projectors and of their expert advisers. Mariemont will always be an interesting museum of American architecture, for probably nowhere else in the United States is there a comparable display of the best work of contemporary architects covering such a wide range of building types.

The visitor to Mariemont is impressed with the work of the architect and of the landscape architect. Less noticeable perhaps to the casual observer, but of no less fundamental importance, are the results accomplished by the town planner and the engineer.

Town Planning

As an example of town planning Mariemont may well be called "a National exemplar" and the broad developments which are now taking definite shape are the result of long, careful, and able town planning. The worth of town planning can best be appreciated by detailed study of the layout both on plan and in the field. Success has met the attempt to conserve the natural beauty of the site and to provide a community which shall be not only attractive but also of maximum usefulness to its citizens.

Engineering

Of particular interest to, and within the province of, the engineer are the utilities of the town, most of which are not seen and may not be appreciated by the visitor. The building of an entire new town on practically unoccupied

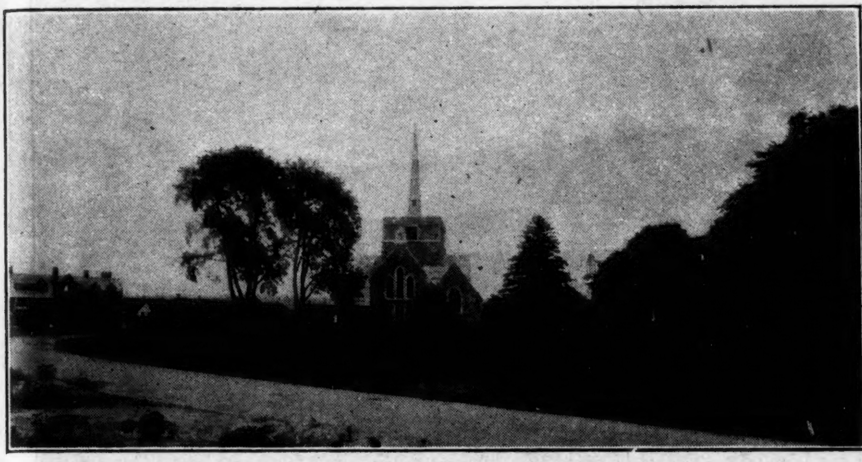

FIG. 2.—THE COMMUNITY CHURCH, MARIEMONT, OHIO.

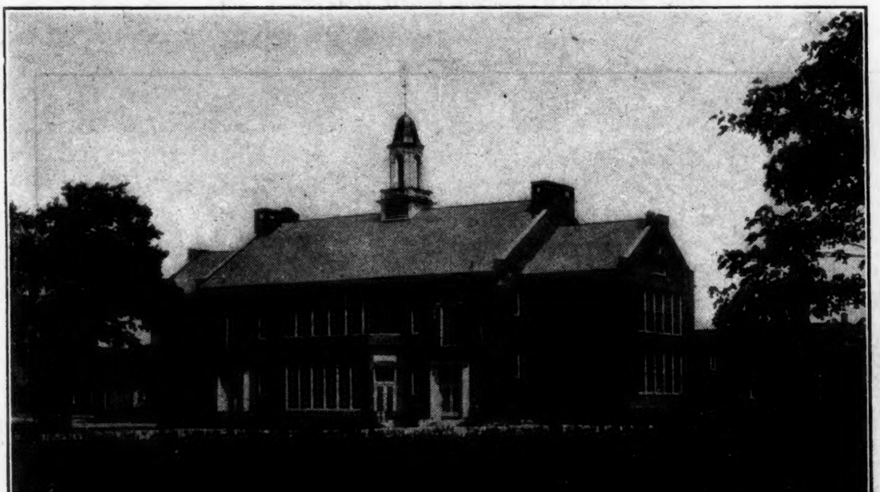

FIG. 3.—THE GRADE SCHOOL, DALE PARK, MARIEMONT, OHIO.

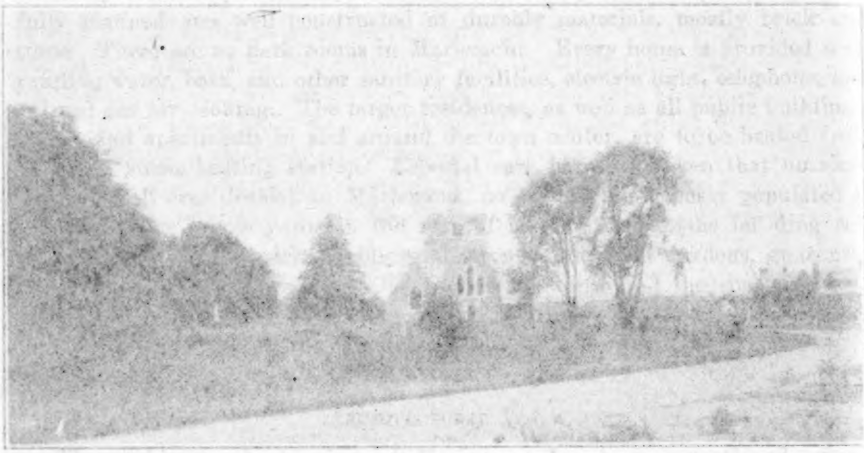

Fig. 2.—The Community Church, Mariemont, Ohio.

Fig. 3.—The Grade School, Dale Park, Mariemont, Ohio

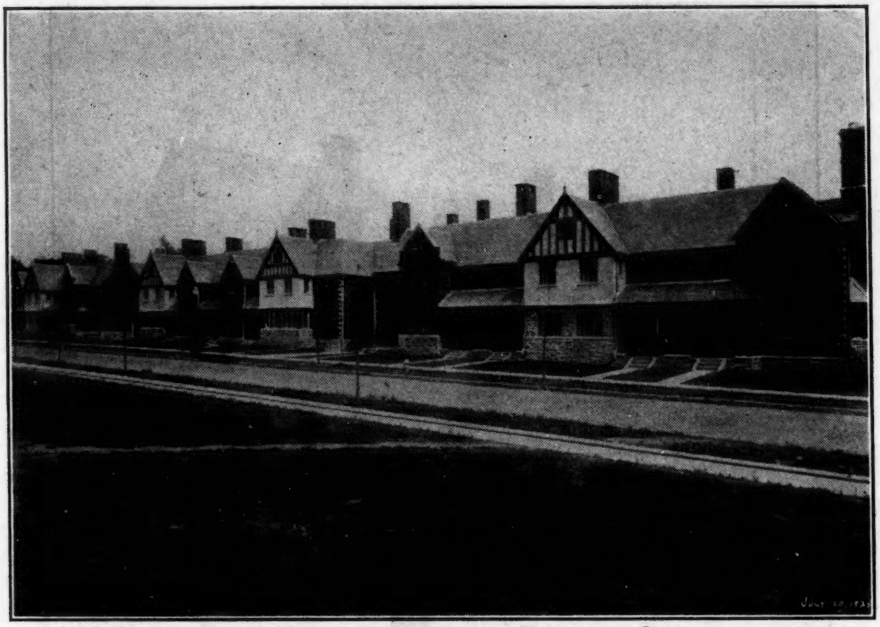

FIG. 4.—TYPICAL GROUP HOUSES, MARIEMONT, OHIO.

FIG. 5.—TYPE OF DUPLEX HOUSES, MARIEMONT, OHIO.

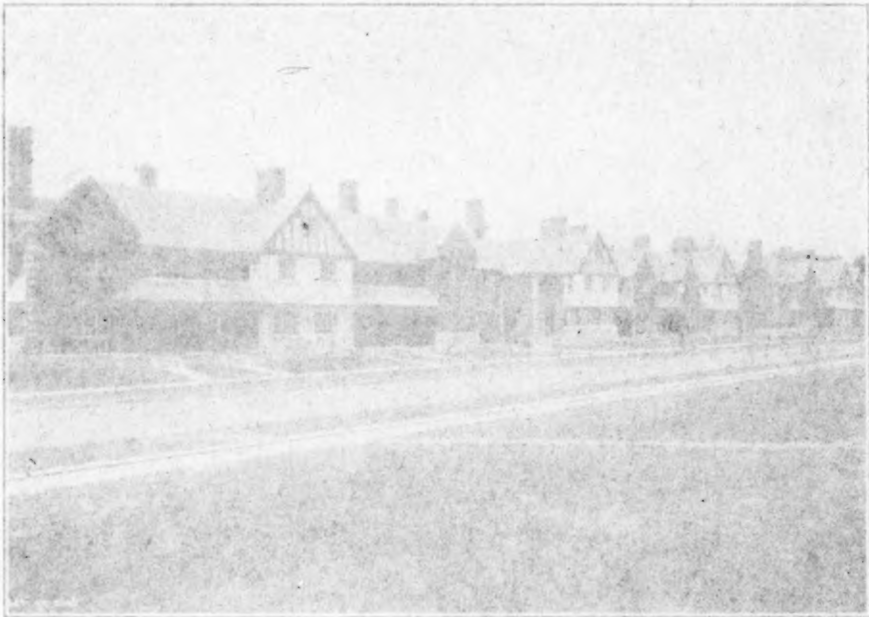

Fig. 4.—Typical Group Houses, Mariemont, Ohio.

Fig. 5.—Type of Duplex Houses, Mariemont, Ohio.

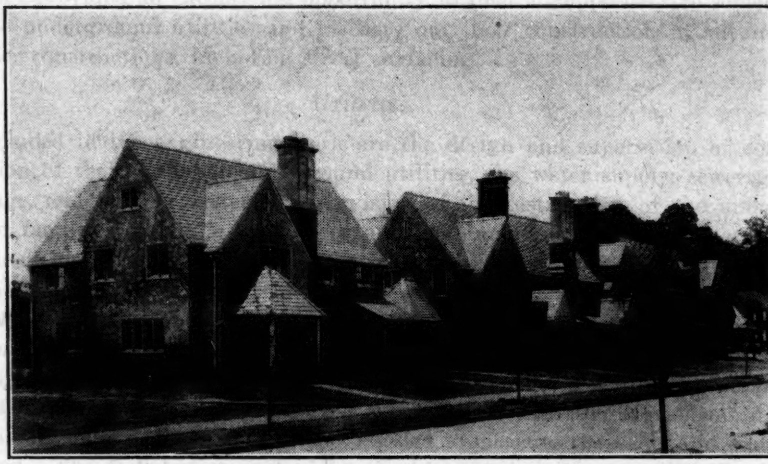

FIG. 6.—SINGLE HOUSES OF SIX AND SEVEN ROOMS, MARIEMONT, OHIO.

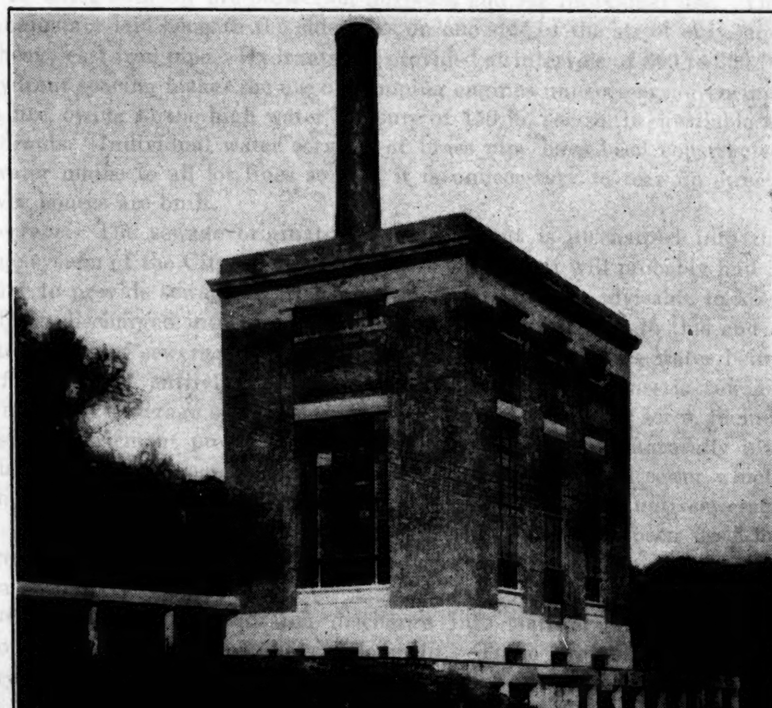

FIG. 7.—BOILER-HOUSE FOR CENTRAL STATION HEATING SYSTEM, MARIEMONT, OHIO.

FIG. 6.—SINGLE HOUSES OF SIX AND SEVEN ROOMS, MARIEMONT, OHIO.

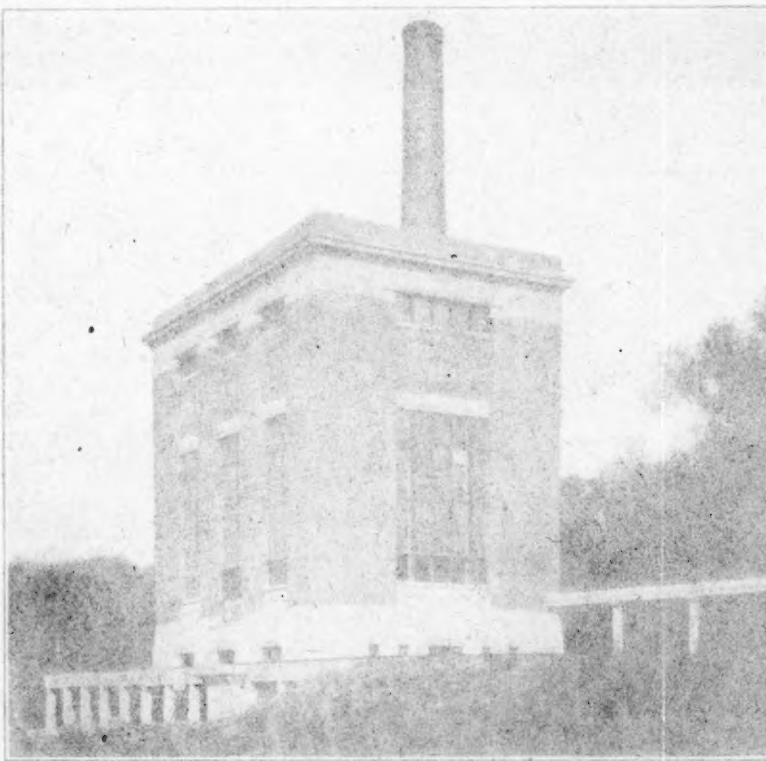

FIG. 7.—BOILER-HOUSE FOR CENTRAL STATION HEATING SYSTEM, MARIEMONT, OHIO.

territory provided an exceptional opportunity to plan a comprehensive scheme for the underground utilities and to carry out their construction in advance of other constructions, including street surfacing.

UTILITIES

Included in the engineering work are the design and supervision of construction of the municipal underground utilities, the water supply, sewerage, drainage, and heating systems; supervision of the installation of the underground facilities of public service corporations, including gas, telephone, and electrical distribution systems; and the design and supervision of street pavements.

Water Supply.—The water supply is obtained through two entirely independent supply mains connecting with the high-pressure water system of the City of Cincinnati. The use of duplicate mains insures a continuous supply of water. Near the junction of each supply main with the Mariemont distribution system, a detector meter is installed to measure the volume of water supplied. The distribution system for the town is built as a "gridiron" with practically no "dead ends", thus assuring a dependable and uninterrupted supply of water both for fire-protection purposes and for individual use. The water mains are laid beneath the sidewalks on one side of the street only, and are of heavy cast-iron pipe. Hydrants are provided at intervals of 300 to 350 ft. This hydrant spacing makes the use of pumping engines unnecessary, even in a serious fire, owing to the high water pressure of 150 lb. per sq. in. available at the hydrants. Individual water services of brass pipe have been constructed from water mains to all lot lines so that it is unnecessary to tear up streets whenever houses are built.

Sewerage.—The sewage originating at Mariemont is discharged into the sewerage system of the City of Cincinnati. As Cincinnati will probably find it necessary to provide sewage treatment in the future, it was advisable to keep the volume discharged into the city system at a minimum and to this end a separate system of sewerage was installed, the storm and surface water being cared for in pipes entirely independent of those carrying domestic sewage. The Mariemont sewerage system serves a total area of about 500 acres, including besides Mariemont proper, adjacent territory which drains naturally into Mariemont. The sewerage system empties into a single outfall sewer which, in turn, discharges into one of the main sewers of the Cincinnati sewerage system. Vitrified clay pipe, with overfilled cement joints, has been used for all sewers except in one instance, where a 10-in. cast-iron pipe siphon crosses under a deep gully.

Drainage.—Storm-water drains discharge into natural watercourses at numerous points throughout the development. These drains are of ample capacity to serve both Mariemont and the entire drainage area tributary thereto. By permitting the storm-water drains to discharge at numerous points a considerable saving has been made in the length of large pipes which would have been needed if both domestic sewage and storm water had been cared for by a combined sewerage system. The storm water from the streets enters the drains entirely through street inlets of the gutter-mouth type, no catch-basins

being used; access to the inlets is provided by a light-weight frame and cover in the sidewalk. At Mariemont the streets will be kept practically free from dirt and other material which might enter the drains and clog them, and as the drains are built to give good self-cleansing velocities no trouble from this source is anticipated. The use of inlets instead of catch-basins not only reduced construction costs but also eliminated the maintenance cost which would have been necessary for cleaning catch-basins. Vitrified clay pipe is used for the smaller sizes of drains and reinforced concrete pipe for the larger drains.

Central-Station Heating.—One of the distinctive features of Mariemont is the central-station heating system. A boiler-house of fire-proof construction, shown in Fig. 7, is located under the bluff at the southeasterly corner of the site adjoining the tracks of the Pennsylvania Railroad and at a level about 100 ft. below that of the main portion of the town. Its equipment includes three Heine water-tube boilers (with provision for a fourth); each boiler has a normal rating of 379 h.p., and is to be operated at 150 to 200% of rating; also, automatic stokers, overhead bunkers, coal conveyors, and other modern devices for the economical production of steam.

At the boiler-house steam is delivered to the main pipes of the distribution system at a pressure of 50 lb. per sq. in. and is available for private consumers throughout the larger portion of the town, as well as for the heating of public buildings. By the use of reducing valves at the individual buildings steam is supplied at low pressure to consumers, the amount being determined by individual meters which measure the condensate. The steam-heating distribution system is along the rear lot lines instead of in the streets.

Work of Public Service Corporations.—A distinctive feature of Mariemont lies in the fact that all utilities are underground. This accomplishment was simplified by the fact that those in charge of the development of the town had to deal with only two public utility corporations, namely, the Union Gas and Electric Company and the Cincinnati and Suburban Bell Telephone Company, the officers of which entered heartily into the spirit of "the Mariemont ideal". As soon as the streets were graded these companies installed pipes and conduits, with the result that everywhere throughout Mariemont there is "service", and has been practically from the beginning. This, with the early installation of water mains and sewers, enabled the construction of the town to proceed rapidly. Fig. 8 shows the relative positions of public utilities in a typical 50-ft. street.

Electricity.—Electric current for both domestic use and street lighting is delivered over a high-tension line to a transformer station, a building expressly erected for this purpose near the northerly edge of the town. This building, of brick and concrete, contains the most efficient equipment and is automatically operated from the electric company's main power plants. Leading from this sub-station are two circuits, lighting and power, and at short intervals along these lines there are transformers which reduce the voltage for domestic use to 110 (alternating current). The entire distribution is carried by underground conduits which lie in the street close to the curb line.

Street Lighting.—The street lighting system of Mariemont has been most carefully planned, the town being zoned for lighting purposes according to best modern practice. The town center will be intensely lighted from twin light standards, 18 ft. high, each unit having one 1 000 c-p. incandescent lamp. These standards, which here are 100 ft. apart and opposite each other, are of cast iron, simple and dignified in design. The same model of standard is used throughout the village, although the type of lamp termination differs according to locality. Main thoroughfares and boulevards are lighted by single-light 600 c-p. units, 15 ft. high, and spaced from 100 to 150 ft. apart in staggered formation. Residential streets have 400 c-p. units which are 12 ft. high and approximately 175 ft. apart. After close study of existing street lighting systems, the officials of Mariemont determined on the use of the Novalux Form 12 lighting unit, with alabaster rippled globes and canopies, for streets and boulevards. Form 25-A Novalux units, each equipped with a 250 c-p. lamp and dome reflector, provide lighting for the service lanes in the rear of the houses. Current for street lighting is distributed by lead-covered cable sunk in the ground only a few inches, thus following the latest urban method.

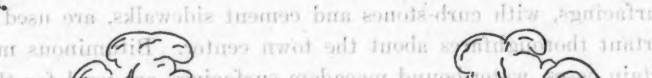

Fig. 8.

Gas.—Gas for Mariemont is brought to the town limits through a high-pressure supply main connecting with the Cincinnati system. The pressure is reduced to normal domestic pressure by a standard high-pressure regulator. The distribution system is built as a gridiron. Service pipes are carried to every lot.

Telephone and Telegraph.—To insure absolute absence of overhead wires, the Telephone Company has installed a complete distributing system of under-

ground conduits in the streets. By this means all telephone and telegraph lines, including trunk lines from Cincinnati to the east, are placed underground. Service connections have been brought to every lot. It is anticipated with the rapid development of the radio there will be no need for aerials.

Street Construction.—Street construction has been carried out in a way to promote both usefulness and economy. As soon as the streets were brought to sub-grade, all underground structures within them were placed, whereupon a temporary surfacing of gravel was laid on the sub-base, this surfacing being adequate to permit the use of streets during building operations and later serving also as a foundation for permanent pavements. The permanent street surfacing is not laid until after the completion of all underground utilities with their service connections to lots, and generally until the buildings fronting on the street have been constructed. Thus, the permanent surfacing, once in place, need not be disturbed, neither is it subjected to the wear and tear consequent on the use of the street by trucks and other heavy vehicles while construction is in progress. Concrete and asphalt roadway surfacings, with curb-stones and cement sidewalks, are used for the more important thoroughfares about the town center. Bituminous macadam and, in certain cases, water-bound macadam surfacings are used for the roadways of the less used residential streets. Service lanes in the rear of buildings are surfaced with concrete.

Project Organization

The development of Mariemont is being carried out by The Mariemont Company, organized by Mrs. Emery and incorporated under the laws of Ohio. The President of the Company is Mrs. Emery's business representative, Mr. Charles J. Livingood. It required several years to obtain the various parcels of property comprising the site before active planning of the development was begun. The town planner, who also handled the landscape architectural features of the development, the several architects, and the writer's firm which was retained for the engineering features of the project, have worked in co-operation for the success of the undertaking which is now being built under engineering direction and supervision, a partner of the engineering firm serving as General Manager for The Mariemont Company.

Municipal Operation and Management

At present, one of the principal sections of the town, the northwesterly section known as Dale Park, is completed and its buildings are occupied by more than three hundred families. To care for the existing population, The Mariemont Company maintains the nucleus of a town organization, including a teaching staff for the school, police force, fire department, and community nurse, together with a force for municipal maintenance work.

In the beginning, all the homes constructed by The Mariemont Company are being rented. Ultimately most of them will be sold, and the town will then be placed in the hands of those who live there to control its destiny.

It is planned to incorporate the Village of Mariemont, with its own town government, a mayor and a council, on the city manager plan. The charter will be based on suggestions and safeguards developed through special studies made by the Rockefeller Bureau of Municipal Research. When the town government is established and in full running order, The Mariemont Company will withdraw. Community buildings, parks, playgrounds, streets, and other features of public nature, will then be presented by Mrs. Emery as her gift to the new town of Mariemont.

THE CINCINNATI CITY PLAN IS NOW LAW*

By George B. Ford,† Esq.

For the first time in the United States, a complete, comprehensive city plan has become the law of a city. The whole plan actually can be enforced, thanks to the exceptionally broad powers granted to the City Planning Commission of Cincinnati, Ohio, by the State laws and the City Charter.

Elsewhere throughout the country, city planning commissions are strictly advisory. They rarely have any power except that in some States they have the right to control the layout of sub-division plats and in some they have art jury powers permitting them to veto the location of public works of art and sometimes to control the appearance of public buildings and structures. The authority of the City Plan Commission of Cincinnati goes infinitely further than this, for under the statute and the charter, there can be no departure from any item of the city plan, once adopted by the Commission, except by a two-thirds vote of the full membership of the City Council, after public notice and hearing, accompanied by the approval of the department head most affected.

Therefore, the problem, which deserves most earnest discussion, is whether city planning commissions throughout the country, and regional planning commissions as well, shall continue to have merely advisory powers or whether extensive powers, such as those granted by the Ohio statute, are really practicable or desirable.

In Massachusetts, for example, the planning boards have no power whatsoever except that their existence is compulsory. The argument for strictly advisory powers is that if the city planning board cannot convince the public and the city officials that its ideas are the best, there must be something wrong with the ideas. The further argument is made that legislative authority should not be divided, but rather concentrated in one body, that is, the city council, which alone is strictly responsible to the electorate. The claim is made that the State Legislature has no right to delegate to a non-elective, non-responsible body any legislative power such as making the city plan the law of the city. In any case, no city council would consent even to sharing the control of the city plan with another body.

In Ohio, however, the claim is made that city planning is a highly specialized and a highly technical matter; that the preparation of a plan that is worth anything, requires the concentrated effort of a selected group of exceptionally

Note.—Written discussion on this paper will be closed in January, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

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† Vice-Pres., Technical Advisory Corporation, New York, N. Y.

intelligent and experienced citizens, aided by the best technical advice. It is maintained that the average city council has neither the time nor the experience necessary to frame an effective city plan, and, in fact, is only too glad to shift the burden, and the inevitable charges of favoritism involved in most planning matters, to another body. It is felt that the making and the promulgating of the city plan is comparable to the making and issuing of health regulations by a health board, or to police or traffic regulations by a police board, or fire prevention regulations by a fire commission. In each of these cases there appears to be little question but that the State Legislature has the right to delegate specific legislative power to these strictly appointive boards. It is merely an extension of the same idea to delegate to a city planning commission the right to frame and enact a city plan, of course, giving the city council a veto power over it, provided this veto cannot be exercised too easily.

The City Planning Commission in Cincinnati consists of seven members with the Mayor as Chairman and with the Director of Public Service and the President of the Park Board as *de facto* members. The Commission contains in its membership, three manufacturers, two lawyers, and one physician of the community.

The power and duties of the City Planning Commission according to the statute are as follows:

"Sec. 4366-2. The powers and duties of the commission shall be to make plans and maps of the whole or any portion of such municipality, and of any land outside of the municipality, which in the opinion of the commission bears relation to the planning of the municipality, and to make changes in such plans or maps when it deems same advisable. Such maps or plans shall show the commission's recommendations for new streets, alleys, ways, viaducts, bridges, subways, parkways, parks, playgrounds, or any other public grounds or public improvements; and the removal, relocation, widening or extension of such public works then existing. With a view to the systematic planning of the municipalities, the commission may take recommendations to the mayor, council and department heads concerning the location of streets, transportation and communication facilities, public buildings and grounds. The commission shall have the power to control, preserve and care for historical land marks; to control in the manner provided by ordinance the design and location of statuary and other works of art, which are or may become the property of the municipality; and the removal, relocation and alteration of any such works belonging to the municipality; and the design of harbors, bridges, viaducts, street fixtures and other public structures and appurtenances. Whenever the commission shall have made a plan of the municipality, or any portion thereof, no public building, street, boulevard, parkway, park, playground, public ground, canal, river front, harbor, dock, wharf, bridge, viaduct, tunnel, utility (whether publicly or privately owned) or part thereof shall be constructed or authorized to be constructed in the municipality or said planned portion of the municipality, until and unless the location thereof shall be approved by the commission; provided that in case of disapproval, the commission shall communicate its reasons for disapproval to council, and the department head of the department which has control of the construction of the proposed improvement or utility; and council, by a vote of not less than two-thirds of its members, and such department head shall together have the power to overrule such disapproval. The narrowing, ornamentation, vacation or change in the use of streets and other public ways, grounds and places shall be subject to similar approval, and disapproval may be similarly over-

ruled. The commission may make recommendations to any public authorities or to any corporations or individuals in such municipality or the territory contiguous thereto, concerning the location of any buildings, structures or works to be erected or constructed by them.

"Sec. 4366-3. The municipal planning commission shall be the platting commission of the municipality, and all the powers and duties provided by law for platting commissioner or commissioners of municipalities shall upon the appointment of a municipal planning commission under this act, be deemed transferred to such commission.

* * * * * * * *

"Sec. 4366-7. The city planning commission of any municipality shall have the power to frame and adopt a plan or plans for dividing the municipality or any portion thereof into zones or districts, representing the recommendations of the commission, in the interest of the public health, safety, convenience, comfort, prosperity or general welfare, for the limitation and regulation of the height, the bulk and location (including percentage of lot occupancy, set-back building lines, and area and dimensions of yards, courts, and other open space), and the uses of buildings and other structures and of premises in such zones or districts."

The powers and duties of the City Planning Commission according to the City Charter of 1918 are almost identical. The City Charter even agrees with the State law in requiring all sub-division plats not only within the city but for three miles outside it to be approved by the Commission before they can be offered for record and before the streets can be dedicated.

Since the City Plan of Cincinnati has been law every question that has arisen as affected by the City Plan, has been decided in accordance with the plan. All city departments are co-operating enthusiastically. In no instance has the City Council over-ruled the City Plan. It is true that several minor modifications have been made in the plan as a result of further study by various city departments. In particular, a more temporary, but more immediate solution of the traffic difficulty at Brighton Corner was adopted by the City with the sanction of the Planning Commission.

During the short period the plan has been operative, a fire-engine house and several schools have been located in accordance with it. The site proposed for the Public Auditorium has been retained for that purpose despite strong demands that it be released for other uses. The Workhouse property has been retained as a playfield site despite demands to the contrary. The traffic and transit difficulties at Peebles Corner are being solved in compliance with the plan. The various down-town roadways are being widened according to the plan despite strong opposition. The street-paving program as well as the Eighth Street Viaduct reconstruction also accords with the plan. The location of a railroad union passenger station and rights-of-way relocation are being carried out according to the plan. New parks and playfields conform with the plan. All the sub-division plats, even for three miles outside the city limits, accord with the plan. Traffic regulations are in accordance with the plan. In general, it may be said that there has been no departure from the plan except in minor instances where the City Planning Commission was convinced that it was possible to improve on it.

The general impression of those who are watching the effect of the Cincinnati method is that it is proving highly successful and is a distinct improvement on the strictly advisory powers of most other planning commissions. It means that the presumption is in favor of the plan because it is the law and also because it was worked out with a great deal of care and thought. It means that the obligation rests with any one who disagrees with the ideas presented to prove that, all things considered, the plan can be improved upon and then the burden rests on him to present a better solution and to convince the City Planning Commission or the City Council that he is right. In other words, it means stability to the plan as a whole, features which do not exist in the same degree in most other States.

It is true that in a city such as Memphis, Tenn., for example, the Mayor and the City Council, on account of their interest in the plan, may virtually give it the force of law that approximates the effectiveness of the Cincinnati plan. This, however, presupposes a strong and continuing interest in the plan on the part of the leading city officials.

In Springfield, Mass., the City Council passed an ordinance that no city official or department should depart in any way from the city plan without first trying to obtain the approval of the City Planning Board, and failing that, the approval of the City Council. This method also has worked well in practice, thanks to the activity of the City Planning Board and the general public interest, although, again, the effectiveness of this method depends on a continuing active interest in the plan on the part of leading public officials.

The Cincinnati method has the advantage of continuing its effectiveness through changing administrations and even over periods of possible public apathy.

The writer has felt that until recently the powers of a city planning commission should be purely advisory but in light of this recent experience in Cincinnati, he is now convinced that if city planning is to be vital in the functioning of communities, the Cincinnati method should be applied generally to city and regional planning. Of course, the success of this method depends on the quality of the planning commission, but the writer's experience in working with more than one hundred different commissions has convinced him that most of the members would measure up to the job. In fact, the very seriousness of the responsibility entrusted to such commissions would inspire them to make the plan a masterpiece in which all would take pride.

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GARBAGE DISPOSAL

A SYMPOSIUM*

Note.—Written discussion on this Symposium will be closed in January, 1927. When finally closed the Symposium, with discussion in full, will be published in *Transactions*.

*** Presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 21, 1926.**

ADMINISTRATIVE AND ENGINEERING WORK IN THE COLLECTION AND DISPOSAL OF GARBAGE: A REVIEW OF THE PROBLEM

Samuel A. Greeley,* M. Am. Soc. C. E.

Synopsis

This paper describes briefly some of the administrative and engineering problems in projects for the collection and disposal of garbage. Technical literature occasionally states that the disposal of garbage has not kept pace with other sanitary engineering works. Such statements are generally coupled with the suggestion that closer adherence to competent technical guidance would greatly improve the results. This is a sound suggestion. Some of the troubles are inherent in the situation and will yield finally only to general public opinion.

The paper outlines relative costs for garbage collection and disposal and other sanitary engineering works. Typical procedures for the acquisition of garbage disposal plants are discussed, recent contracts and specifications outlined, and engineering items in garbage disposal listed.

General Statement

This paper relates briefly to some of the present-day administrative and engineering problems in the development of projects for the collection and disposal of garbage. Occasionally statements appear in technical journals and engineering society proceedings that the disposal of garbage has not kept pace with other enterprises dealing with the public health and comfort, commonly termed sanitary engineering works. Such comparisons inevitably involve remote influences as well as intimate reactions. Criticisms are generally coupled with the suggestion that closer adherence to competent technical guidance would greatly improve the results; which is, of course, a sound statement. Some of the troubles are inherent in the situation and usually will yield only to general public opinion.

Works for the collection and disposal of garbage involve much lower per capita construction costs and require a much larger personnel in operation than other sanitary engineering works, such as water supply and sewerage. Thus, an opportunity is relatively easily acquired to employ a comparatively large number of unskilled workers. Collection equipment, and even garbage disposal plants, can usually be financed without great infringement on the public credit and, therefore, often without the necessity of general public attention and support. Inherently, therefore, the interest in the collection and disposal of garbage is measured by different yard-sticks than those applicable to other sanitary engineering works.

* (Pearse, Greeley & Hansen), Chicago, Ill.

Thus, in one community where the writer has been engaged for the last decade, garbage disposal works are estimated to cost about \$75 000, whereas \$2 500 000 has been spent on an additional water supply and \$2 000 000 is being spent on sewage disposal.

A number of other less fundamental troubles may be briefly stated:

First.—Garbage disposal works proper have been sometimes acquired by municipalities under general service contracts or by purchase under general specifications which require the fulfillment of certain guaranties. Thus, the design, as well as the cost, becomes competitive, and improvements in the design resulting from experience are confined by the exigencies of a somewhat promiscuous competitive field. Manufacturers of garbage disposal equipment of the better class are restricted in their developmental work by the hazards of the competition of low-priced and low quality equipment.

Second.—The selection of a method of garbage disposal from several satisfactory processes is occasionally left to a consideration of competitive bids received under specifications which define the work to be done, but which leave both the choice of the process and the details of design to each bidder. Under such circumstances the selection of the "best" bid is not easy and the choice of a process, as well as the quality of design included in the bid, are likely to be oriented somewhat to the limits imposed by the competitive conditions. Such a procedure should not be confused with the general service contract under which the city does not acquire the plant, but contracts for the service of garbage collection and disposal.

Third.—The award of a contract for a garbage disposal plant generally leaves the final acceptance of the work to the fulfillment of certain guaranties to be determined by tests. Frequently, the making of satisfactory tests is hard because of the difficulty of accumulating a sufficient supply of the desired quality of garbage and rubbish. The value of the tests as an index of normal operating efficiency may be questioned because test runs are sometimes made with specially skilled labor. The rejection of a plant on the part of a municipality is likely to mean not only litigation, but also a delay in the solution of the garbage disposal problem. Hence, even if the rejection is upheld in the Courts, there is a large and undesirable economic loss. Uncertainties such as these tend to introduce a gambling element which may attract an undesirable type of contractor, and thereby work against safe and conservative design and construction.

The purpose of this paper is to throw some light on these items. Broadly speaking these inherent troubles are limited more or less directly in accordance with the amount of skilled and experienced engineering effort applied to the problem. Such engineering work may be, and often is, supplied by general contractors, by city engineers and other municipal officers, and by consulting or practicing engineers. Generally, the quality and freedom and not the source of the engineering work are important. Improvements in the garbage field are most likely to come from a better understanding, on the part of the general public, of the problem and its inherent difficulties.

Cost Aspects of Collection Work

The collection of garbage is largely a problem of administration and organization. The engineer is concerned chiefly with the collection equipment such as wagons, motor trucks, trailers, stables, and garages, with considerable opportunity for productive work in the selection of equipment and the subsequent routing and operation. The actual design of wagons and trucks, however, is done largely by the manufacturers, so that finished products are commonly bought under general specifications. Very often such specifications and methods of purchase do not permit a close and actual money comparison of bids.

In a city of several hundred thousand people, the equipment for garbage collection may represent an initial investment of perhaps 30 to 40 cents per capita. As compared with this, works for the collection of domestic sewage may cost for construction, one hundred times as much, or \$30 to \$40 per capita, depending on the concentration of population, the topography, and similar features. The operation of the garbage collection equipment, however, is much more costly than the maintenance and operation of a sewerage system. A city of 400 000 people may need 50 to 60 garbage collection units (exclusive of ashes and rubbish), with 125 to 150 workmen. Sewer maintenance requires probably less than 25 workmen. Thus, a relatively low cost equipment can be secured, which provides work for a large number of men.

Cost Aspects of Disposal

The disposal of garbage involves administrative and engineering problems. The design and construction of a garbage disposal plant is essentially a problem involving both civil and mechanical engineering. In a city of several hundred thousand, the construction cost of a complete garbage disposal plant may be from \$0.75 to \$1.00 per capita. For comparison, the cost of a complete sewage treatment plant is likely to be ten times as much. The operation of the garbage plant, however, will require three to four times as many men as might be required in the operation of the sewage treatment plant. In projects for the disposal of garbage, therefore, as compared with the disposal of sewage, the first cost is relatively low, and the operating cost and personnel are relatively high. Much the same comparison holds between garbage works and other municipal enterprises, such as water supply, paving, etc. The collection and disposal of garbage is one of the least costly municipal enterprises to acquire and one of the most expensive to operate. This emphasizes the administrative aspects of the work, as compared with the engineering.

Relative Costs of Collection and Disposal

Not only is the collection and disposal of garbage more expensive in annual operation than most other municipal enterprises, but the collection part of the work usually involves by far the larger cost and greater personnel. Of the garbage budget 75% may easily go to the collection work. As the collection of garbage is so largely administrative, its greater importance on a cost

basis tends to detract from the consideration due the engineering aspects of the work as a whole.

Present Practice as Regards Incineration

At present, incinerators for garbage disposal are generally acquired on a "duty" or performance basis. After funds are available and a site has been selected, and perhaps acquired, the city advertises for bids on a general specification requiring each bidder to submit his own plans. Occasionally the specifications are brief, stating only the capacity desired and perhaps the height of the chimney. More extensive specifications cover the materials and include guaranties of operation. More recently, plans have been prepared detailing the general layout, the building construction, the chimney, the approaches, and similar items. This limits the bidders' plans to the incinerator proper, reduces the items not easily compared on a money basis, and thus tends to a more equitable letting. Sometimes the type of incinerator is specified, that is, whether it shall be of the inside or outside storage type, and the limiting areas of the grates, flues, combustion chambers, and other parts are stated. In some specifications, only incinerators with a service record of a stated period of years are permitted. This tends to limit the field.

The specifications used in Milwaukee, Wis., in 1909 required the bidders to guarantee an efficiency and an operating cost, and the bids were compared on a computed annual cost. Thus the service character of the project was frankly embraced. More recent specifications have not included this feature, perhaps because of the difficulty of measuring practical operating costs under test conditions.

A report outlining such general specifications has recently been prepared by the Committee on Refuse Collection and Disposal of the Sanitary Engineering Section of the American Public Health Association.* Such reports are helpful in promoting the detailing of the incinerator specifications to the utmost under present competitive conditions and patent limitations.

Outline of the Incinerator Specifications

Under some specifications for a garbage incinerator recently prepared by the writer, bids were asked for under five items as follows:

(*a*) Incinerator furnaces with all appurtenances;
(*b*) Incinerator building and scale;
(*c*) Chimney;
(*d*) Runway; and
(*e*) Sewers and sewage disposal plant.

This division into items separated the incinerator furnaces from the other construction items, and thus facilitated the canvassing of bids and the selection of a contractor. With this and other considerations in mind, it was felt reasonable to allow monthly estimates amounting to not more than 70% of the value of the work properly performed at the time of the estimate. The

* *American Journal of Public Health*, April, 1926, Vol. 16, No. 4.

specifications were then developed in accordance with the following general outline.

Description.—The work to be done was described as the construction, testing, and placing in operation of the incinerating plant. The description included definitions of garbage and rubbish, the proportions of materials as collected, together with analytical data as a basis for guaranties and tests.

Divisions.—Bids were asked for under Divisions *A* and *B*. Both divisions required a complete incinerator plant, including the runway, building, furnace, chimney, sewers, sewage disposal plant, and all necessary appurtenances. Division *A* allowed the bidder considerable latitude in the design and arrangement of the plant. Division *B* required certain features considered desirable by the city and its engineer.

Drawings.—Each bidder was required to submit with his proposal, drawings in sufficient detail to show the construction of the various items comprising the furnace and appurtenances. Before starting work, the contractor was required to submit detail drawings for the entire work. These drawings were to be approved by the city officials and their engineer and the work was required to be built in conformity with the approved drawings.

Statements.—In addition to the usual statements required covering the amount of power, the schedule of labor, the average temperature, the capacity, and the like, the specifications required a list of other plants then in operation to which the bidder referred as examples of his construction and which the city officials and their engineer could visit and investigate so as to satisfy themselves that the proposal could be carried out. It was stated that the city officials desired the information to clearly support the proposal of the bidder. Therefore, the statement relating to incinerators similar to the one proposed was required to be complete and to include features in which the design of the operating incinerator differed from the design of the proposed incinerator.

Guaranties.—All tests and failures were covered in much the same manner as in other similar specifications. The bidder was required to state the amount, if any, of additional fuel he considered necessary.

Workmanship and Materials.—Fifty-seven paragraphs were included on the workmanship and the quality of construction materials.

Items.—Sixty paragraphs were included to describe the five items of the work. Every effort was made to describe the items in such complete detail that the bid price would largely control in the selection. This was not entirely possible for the furnaces proper because of the variety of designs which might be submitted under the two divisions of the specifications.

General Conditions.—The usual general conditions, including thirty-four articles, covered all the general requirements of the work.

These specifications were accompanied by a set of four drawings indicating the location and general arrangement of the plant. Perhaps the special feature of these specifications was the requirement of references to operating incinerators similar to those proposed. As a matter of fact, the city officials and their engineer investigated, by actual visit, operating plants similar to those proposed by several of the bidders.

Present Practice as Regards Reduction

Most reduction plants have been built and developed by contractors having general agreements with cities for the service of garbage disposal. Garbage reduction works which are now in operation have been built by contractors at Boston, Mass., Pittsburgh, Pa., Baltimore, Md., Cincinnati, Ohio, Detroit, Mich., and other smaller cities. (See Table 1.) Plants taken over by cities, but originally built by contractors are those at Chicago, Ill., Philadelphia, Pa., Cleveland, Ohio, and Washington, D. C. The Cobwell plants at Los Angeles, Calif., and New York, N. Y., were designed and built by contractors, but they are not now in operation. The reduction works at Columbus, Ohio, and Rochester and Schenectady, N. Y., were built by the cities under more or less detailed plans and specifications. The plant at Syracuse, N. Y., was built by the manufacturer under a special agreement with the city and was later operated by him for a time.

Changes and improvements have been made in many of these projects. In some of the plants, odor problems and other operating defects have been encountered, and details for improvement have been designed. Much of this detailed experience is not available for general use in developing designs. Work of this character has been done at Detroit and Rochester and should develop interesting data. However, general design data are not on the whole as accessible as for sewage treatment and water purification works.

TABLE 1.—Administrative and Engineering Work in the Collection and Disposal of Garbage. Data Relating to Reduction Plants.

Plant.	Originally built by:	Now operated by:
Baltimore, Md.	Contractor	Contractor
Boston, Mass	Contractor	Contractor
Chicago, Ill.	Contractor	City
Cincinnati, Ohio	Contractor	Contractor
Cleveland, Ohio	Contractor	City
Columbus, Ohio	City	City
Detroit, Mich.	Contractor	Contractor
Indianapolis. Ind.	Contractor	City
Philadelphia, Pa.	Contractor	City
Pittsburgh, Pa.	Contractor	Contractor
Rochester, N. Y.	City	City
Schenectady, N. Y.	City	City
Syracuse, N. Y.	City	Contractor
Washington, D. C.	Contractor	City

Except for the relatively few reduction works built under city plans and specifications, most of the plans in the larger cities of the United States have been built, as far as the municipality is concerned, under general specifications requiring certain standards of service. The contractor has been left free to design and detail the work proper. Many of the larger plants so built were placed in operation twenty to thirty years ago when their design was less advanced than at present. Many plants were developed by inventors and have since been renewed or rebuilt with more or less standard manufactured equipment. Consequently, the engineering work has been somewhat limited to the assembling into working units of a variety of readily purchasable

mechanical parts. This procedure has tended to develop a practice in the construction of reduction works akin to the purchase of boilers, engines, piping, generators, etc., in a power plant. Under conditions usually obtaining, it is perhaps likely that such purchases can be made more advantageously and the entire plant better assembled by a private organization than by a municipality. However, in both cases the quality of engineering work will generally establish the success of the project.

Present Practice as Regards Hog Feeding

Garbage disposal by feeding to hogs is a development from the small cities under more or less informal agreements for collection and disposal. Few detailed plans and specifications have been prepared, with the exception recently of the Los Angeles Farm. The engineering work in the larger projects should include facilities for receiving and distributing the garbage, hog houses and pens, roadways and platforms, water supply, and sewerage, together with adequate methods for disposing of unconsumed garbage. Where proper design of such details is included in the construction of a hog farm, disposal by feeding to hogs is not as profitable as under a more informal procedure, except perhaps in large undertakings, but is less likely to create a nuisance.

TABLE 2.—Administrative and Engineering Work in the Collection and Disposal of Garbage. Engineering Items in Garbage Disposal Works.

Incinerators.	Reduction plants.	Hog farms.
Buildings and grounds	Buildings and grounds	Buildings and grounds
Unloading facilities	Unloading facilities	Unloading facilities
Storage arrangements	Storage arrangements	Distribution arrangements
Charging apparatus	Raw garbage conveyors	Roadways
Furnace arrangements:	Digesters	Hog houses
Structure	Dryers	Hog yards
Brickwork	Presses	Feeding platforms
Flues	Grease extractors	Fences
Grates and grate setting	Naphtha storage	Farm equipment
Ash and clinker removal	Grease refining	Final disposal of residues
Ventilation	Grease storage	Water supply
Stoking doors	Screens	Drainage
Tools	Crushers	Sewerage and sewage disposal
Combustion chamber	General conveyor system	Facilities for vaccination
Forced draft	Tankage storage	Facilities for farrowing
Dampers and draft control	Shipping facilities	Farm buildings
Chimney	Scrubbers	Silos
Approaches	Floor and floor drains	Auxiliary feed storage
Foundations	Office and laboratory	
Operating facilities	Chimney	
Recording instruments	Settling and skimming tanks	
Water supply	Boiler plant	
Sewerage and sewage disposal	Power plant	
	Foundations	
	Water supply	
	Sewerage and sewage disposal	
	Naphtha recovery plant	

Engineering Details

Table 2 represents a number of the items which indicate the engineering aspects of garbage disposal projects. This list is only general and approximate. Many details are of great importance, such as the design of furnace

bracing and fire-brick arches in the incinerators, and the materials used for piping, valves, and other metal structures subject to corrosion in reduction works. The general arrangements for the economical operation of reduction works are important, including the delivery and use of steam, the control of naphtha losses, and similar items affecting the over-all costs of operation.

Of special importance are the works needed for satisfactory odor control. This aspect relates to the broad considerations resulting in the selection of the method of disposal, and then to the details of odor control, such as the capacity of the incinerator grates, flues, and combustion chambers, and the confinement and treatment of odorous gases at reduction works.

It should be apparent that much operating experience is necessary to secure a sturdy and economical design. In some instances where the hazard of litigation over nuisance is forced into the situation, a careful weighing of the advantages of the different types is required, involving consideration of the first cost of plant construction, the cost of collection and haul as affected by disposal plant location, the net operating costs, and the cost and feasibility of adequate odor control.

Test and Routing Operating Data

Generally, the best design results from a familiarity with adequate, correct, long-time operating experience and data. As a basis for design, such operating data must be specific. Desirable information includes the following items:

A.—Characteristics of the Garbage.—

Quantities of Garbage:
- Seasonal variations,
- Daily quantities,
- Influence of climate.

Physical Characteristics of Garbage:
- Free water,
- Foreign material,
- Age at disposal plant.

Chemical Characteristics of Garbage:
- Moisture,
- Carbon,
- Ash,
- Volatile matter,
- Grease,
- Protein,
- Nitrogen,
- Potash,
- Phosphates.

B.—Incinerator Data.—(All related to the characteristics of the garbage):

Rates of Charging and Burning:
- Fresh and after storage.

Temperatures in:
- Furnace chamber,
- Combustion chamber,

Temperatures in—(*Continued*):
- Flues,
- Chimney,
- Ash-pits.

Forced or Natural Draft:
- Pressures,
- Volumes,
- Temperatures.

Analyses of Products:
- Flue gases,
- Ashes and clinkers.

Economies of Additional Fuel.
Effect of Different Methods for Limiting Radiation Losses.
Structural Strains and Stresses as Related to Life and Design.
Preliminary Dewatering and Drying.
Combustion Chamber Operation in the Removal of Dust and Paper.

C.—Reduction Plant Data.—(All related to the characteristics of the garbage):

Effect of Age of Garbage on Recoveries.
Floors and Floor Drainage.
Rates of Operation of Driers:
- Water evaporated per hour at different temperatures,
- Coal required,
- Temperature and volume control,
- Vapor pressures and moisture absorption,
- Drier gas volumes,
- Analyses of drier gases,
- Losses of recoverable materials,
- Odor control of drier gases.

Design of Digesters:
- Operating data with different shapes,
- Steam economies,
- Volume and treatment of vent gases,
- Design of linings,
- Loading and discharge facilities,
- Value of stirring,
- Drainage features,
- Economical period of digestion.

Design of Percolators:
- Loading and discharge facilities,
- Inlet and distribution of steam,
- Drainage,
- Grease recovery,
- Condensers,
- Naphtha cycle.

Design of Presses:
- Condition of the garbage,
- Time of pressing,
- Feeding or charging the presses,
- Value of grease as compared with cost of pressing,
- Materials of construction.
- Drainage.

The foregoing list is suggestive rather than complete and a reasonable perspective must be taken regarding the precision of the data as affecting economies in the design. Attention should be given to the heat, water, and naphtha balances for the plant as a whole and for the several groups of equipment. Grease-skimming basins and the proper disposal of liquid residues are important. For some of the items too great precision is not warranted, but in the past the writer believes engineers have erred by not securing sufficiently precise records.

In the incinerator field, a great many data are available. Extensive tests, including analytical data, have been secured at Staten Island, New York, by Fetherston, at Milwaukee by the writer, and recently by other observers. Long-term operating records are not so complete and are few in number compared with the analytical data kept and published in the operation of water and sewerage works.

Less extensive operating data are available for reduction works, although the indications are that more are becoming public. Such information would clarify the design of driers, presses, and other equipment as regards operation, and would indicate plant balances in the economical use of coal, steam, water, naphtha, power, and labor.

There is need for more information and for a wider application and use of the experience already had. Work somewhat analogous to the extensive tests by municipalities and sanitary districts of various sewage treatment processes would be very helpful if applied in garbage disposal projects. Extensive sewage experiment stations have been operated in New York, Chicago, Philadelphia, Boston, Cleveland, Milwaukee, Akron, Gloversville, N. Y., Decatur, Ill., and elsewhere and in some of the State Universities. Similar tests of garbage disposal methods might best be undertaken at the larger plants somewhat as has been done at Detroit and Indianapolis. First, complete records would be established followed by special tests of several units and types of equipment. From time to time new kinds of machinery and equipment would be installed and tested. As compared with sewage works, relatively short tests would suffice as the processes involved are generally not biological. Such efforts should be characterized by accurate technical work, care in recording results, and the publication of the data and the conclusions.

The Patent Situation

Many of the various processes for garbage disposal are, or have been, covered by patents. The validity of these patents has never been tested as far as the writer knows. Some of them, such as those covering the Cobwell process, appear to be broad and general, whereas others relate to minor devices. There are several different "makes" of incinerator which seem to be quite similar in general design, but which are apparently carefully guarded by separate patents. Just what constitutes immunity from patent litigation in the garbage field is difficult to express. The present state of affairs, however, probably tends to cloud the general situation and to retard a more rapid development based on fundamental considerations.

Typical Steps in the Acquisition of a Garbage Plant

It is of interest to picture the procedure now followed by many communities in the acquisition of a garbage disposal plant. Action may be prompted by difficulties arising over some informal method of disposal, as by dumping or by feeding to hogs. Increased population and the encroachment of critical districts may require relocation of existing disposal facilities. Occasionally, the fault lies more with the collection than with the disposal.

When such conditions arise, city officials, sometimes with interested citizens, plan trips to other cities where they inspect garbage disposal plants in operation. Much information is furnished by representatives of manufacturers of different garbage disposal plants, and by other cities. The method of garbage disposal is selected, and proposals are invited so as to encourage competitive bidding. After the bids are received a comparison is made, necessarily on the basis of likely performance as well as on cost. A selection having been made, the contract is awarded and steps are taken to start construction.

At this point, injunction proceedings are frequently started to prevent the carrying out of the work. Sometimes objection is made to the proposed location. In other instances the method of taking bids and awarding the contract is questioned. Such proceedings are sometimes long drawn out and quite expensive. In many cities, unless fraudulent practice has been proven, the Court decision favors the municipality and the work proceeds.

When construction is completed tests are required. Occasionally the delays noted have thrown the completion of construction into a new administration and difficulties arise over the final testing and acceptance of the work. Such difficulties are sometimes actual in that the plant itself can not meet the guaranties, and occasionally result from failure to give the plant a fair trial in operation.

If, however, the plant finally comes through these various stages it is more than likely to give favorable results in operation, particularly during the first few years. Many plants of faulty design are giving fairly good results in operation particularly where the collection service is good.

It seems to the writer that difficulties similar to those noted are less likely to occur if the project from the beginning to the end receives consistent, unhampered, and careful engineering work.

Limiting Procedures

The general practice in acquiring garbage disposal works has already been stated. There are limiting procedures which are worth describing.

One extreme includes the preparation of complete detail plans and specifications on which bids for construction can be secured. This means that the city itself would assume responsibility for the design and would proceed in the usual way to select a contractor for building the work. The detailed plans and specifications might be prepared by engineers acting for the city, or by experienced general manufacturers capable of making plans and specifications suitable for competitive bidding.

The other extreme comprises a contract for collection and disposal in which the work to be done is clearly stated and the methods are left to the successful contractor. The recent procedure at Kansas City, Mo., is along this line. The notice to bidders in Kansas City stated that:

"The work for which proposals are invited includes the complete and satisfactory collection and disposal of the garbage of Kansas City for a term of 10 years in a clean and sanitary manner, and the furnishing of all necessary plans, statements, equipment, buildings, plant, apparatus, sidings, land, labor, horses and appurtenances."

Throughout the specifications and contract, the work is consistently described as the service of collecting and disposing of garbage. As far as possible, this service was clearly and fully described. The following list of items from the specifications indicates the topics covered:

1.—Description of work to be done.
2.—Definition of garbage.
3.—Extent of garbage collections.
4.—Collection districts and headquarters.
5.—House treatment for garbage.
6.—Weighing garbage.
7.—Frequency of collections.
8.—Hours of garbage collection.
9.—Emergency collection service.
10.—Complaints.
11.—Regulations for sorting.
12.—Handling house receptacles.
13.—Maintenance and cleaning of garbage equipment.
14.—Painting collection units.
15.—Drainage of garbage.
16.—Transfer stations and equipment.
17.—Methods of garbage disposal.
18.—Disposal of residues.
19.—Inspection by the city.
20.—Workmanship and materials.
21.—Sanitary operation.

The specifications stated that the garbage could be disposed of by any method shown to have a satisfactory record of operation elsewhere. Among such methods, feeding to hogs, processes which recover saleable products, and incineration were included. The contractor was required to provide ample capacity and facilities so that the disposal plants could be operated in a clean and sanitary manner free from nuisance. Among such facilities the specifications included concrete floors, drains, water supply, housing, etc.

Particular attention was directed to the disposal of residues. The specifications stated that such residual products must be properly handled, if the disposal was to be complete, and listed the following:

(*a*) Unconsumed garbage at hog farms.
(*b*) Manure at hog farms.
(*c*) Ashes from incinerators.
(*d*) Sewage, including floor washings, scrubber wastes, grease liquors, etc.

(*e*) Gases from driers and incinerators.

(*f*) Tins, tailings, and rubbish from recovery processes.

(*g*) Tin cans and tinware.

A contract for this work has been awarded and in operation since the early summer of 1925. The bid price was $6.45 per ton for collection and $1.00 per ton for disposal. Under the contract temporary methods of disposal were permitted and these are still in use. A first-class collection service has been developed. Recently, from about 100 000 houses there have resulted about 25 complaints of non-collection service per day. Practically all complaints received before noon are handled by the emergency truck the same day. Complaints received during the afternoon are handled the following morning. Considering the background of garbage collection experience in Kansas City, this evidences good collection service.

Present Indicated Practice

Each project should, of course, be studied in the light of as much operating information and experience as possible. On this broad basis, the following steps generalize the best procedure indicated at present for a municipality in the acquisition of a garbage disposal plant:

a.—A choice should first be made between municipal and contract operation. If contract operation is selected, the specifications and contract should cover in detail the service to be rendered and the standards of operation required, as described for Kansas City. If municipal operation is selected, the procedure is somewhat different as indicated by the following items.

b.—The collection problem should first be studied and developed in sufficient detail to insure satisfactory service to householders, reasonable economy, and proper co-ordination of the collection and the disposal.

c.—A site or sites for disposal plants should be acquired which suit the collection service, and which can be clearly established as integral parts of a sound general plan for garbage collection and disposal for the community as a whole. This general plan must be sufficiently sound to survive injunction proceedings.

d.—The method of disposal best suited to local conditions and available sites should next be determined. The method selected should be defined as far as possible to promote a ready comparison of bids and yet to encourage competition.

e.—The City should then prepare detailed plans for the grounds, approaches, buildings, chimneys, and other appurtenances including the general arrangement of equipment, such as furnaces, boilers, driers, digesters, storage facilities, etc. These plans should be detailed to the greatest extent possible with due regard to maintenance of competition in the equipment field.

f.—The drawings should be accompanied with complete specifications in general accord with the arrangement and topics described on pages 1645 and 1646. These specifications should reasonably and justly protect both the city and the contractor by the elimination as far as possible of uncertainties and hazards.

g.—The specifications as outlined will require reference by equipment bidders to works now in operation similar to those offered in the proposal. Such operating equipment for the two or three more favorable proposals should be inspected and tested as operating units before selecting the most favorable bid. Such a procedure will temper the bids to a reasonable basis and will enable a fairer selection where the selection has to be made on a performance basis.

h.—Some plans detailing the equipment offered will be included with the proposal. Other construction drawings will be required of the contractor. All such plans should be carefully checked and approved by the engineer prior to and during construction.

i.—Construction should be carried out under careful engineering supervision and inspection. Some failures may be charged to poor construction work.

j.—After the plant is built, careful and fair operating tests should be made before acceptance. It is likely that recognition on the part of bidders that acceptance tests are to be made by competent, fair-minded engineers will promote safer and better bidding and will eliminate part of the contingent item in the bid.

k.—After the work is finally accepted, a satisfactory operating routine with adequate records should be established.

This procedure will be better followed if it is entrusted to experienced, disinterested engineers. Probably the best way to obtain results is through the development of a public opinion which understands the administrative and engineering aspects of the collection and disposal of garbage.

GARBAGE COLLECTION AND DISPOSAL, LANSING, MICHIGAN

By Edward D. Rich,* M. Am. Soc. C. E.

Prior to 1916 the garbage of Lansing, Mich., was collected by private contract involving a weekly charge to each household taking the service. In November, 1916, the citizens voted for a general collection to be paid for out of the annual tax levy.

Collection

With this change, the can system of collection was adopted, the City furnishing the cans. Motor trucks are used exclusively for collecting the cans which have a capacity of about 1 bushel and are tapered, being smallest at the bottom. The 2½ to 3-ton truck has been found to be the most satisfactory and economical although various sizes, from ordinary Fords to 3-ton trucks, are in use.

Calls are made daily at hotels and restaurants and weekly at residences. In making the collections, the can covers are not collected, as without the covers, the cans may be nested on the truck, thus consolidating the load. If this nesting is done carefully, no garbage is spilled on the ground or from the truck. The trucks are hosed after each trip.

It is believed that tank-wagon collection would be cheaper than the can system but considering the advantages of efficient and satisfactory service to the householder and also the condition of the garbage, the additional expense, if any, seems justifiable. The elimination of the disagreeable task to the housewife of washing garbage cans is a distinct service that is highly appreciated. The use of miscellaneous receptacles, which are so common if garbage is collected in tank wagons results in deterioration of the garbage before collection, thus rendering it less suitable for feeding purposes.

The City has been unusually successful in obtaining separation of tin cans, glass, crockery, and other foreign materials from the garbage, accomplished largely through newspaper publicity. The people are asked occasionally not to jeopardize their own hogs by putting anything into the garbage that will injure them.

A charge of $1 per year is made by the City for each can in use, the money being kept in a separate fund for maintaining the supply of cans. The wisdom of making any direct charge to the householder is doubtful; the object sought in garbage collection is a clean city and any fee, however small, tends to deter some from taking the service. The cost of collection should be provided for in the tax budget and inspectors should be employed to see that every household avails itself of the service. The percentage of families who pay the can fee indicates that this charge has not had a very serious effect on the extension of collection over the entire city, but there seems to

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be no logical reason why the cost of maintenance of cans should not be paid out of the tax levy the same as the other items of collection cost.

The amounts received from can fees for four years are as follows:

1921	$ 8 403.95
1922	$11 058.90
1923	$11 675.14
1924	$12 884.25

It is stated that the cost of supplying the cans in use for the four years about equals the receipts from can fees. In some years there has been a deficit and in others a profit. A reserve supply of about 1 000 cans is kept on hand.

The cost of collection for four years is shown in Table 3.

TABLE 3.—COST OF GARBAGE COLLECTION, LANSING, MICH., 1921-1924.

Year.	Cost.	Population.	Patrons served.	Population served.	Percentage of population served.	Cost. Per capita cost to total population.	Per capita cost to population served.	Cost per patron.
1921.................	$36 245.54	61 242	7 600	31 540	51.4	$0.59	$1.15	$4.78
1922.................	41 869.03	63 852	8 500	35 275	55.3	0.66	1.18	4.93
1923.................	44 004.06	65 871	11 120	46 150	70.0	0.67	0.95	3.96
1924.................	44 363.07	67 091	11 800	48 970	73.0	0.66	0.91	3.76

These costs include labor, repairs to equipment, gasoline, salary of the superintendent, his office expenses, interest at 6% on the value of five trucks at $2 000 each, and depreciation on these trucks at 14 per cent. The population of the city since the 1920 Census has been estimated in proportion to its growth between 1910 and 1920. No exact record of the number of patrons is kept The figures shown in Table 3 were estimated and are regarded as substantially correct. The 1920 Census gives a relation between the population and number of families in Lansing of 4.15 persons per family. This factor multiplied by the number of patrons gives the estimated population served as shown in Table 3. This method of estimating population served is perhaps not very accurate, but no better means is available.

From the marked increase of patrons in 1923 over 1922, it will be noted that the cost per patron, or per capita of population served, decreases as the percentage of population served increases.

DISPOSAL

During 1916 a municipal piggery was established on a relatively small tract of land owned by the City. This property was not large enough to permit the keeping of sufficient hogs to consume all the garbage collected and nuisances resulted. During the latter part of 1921, a farm of 120 acres was purchased and the development of the present feeding plant begun. This

farm is $4\frac{1}{2}$ miles from the center of the city, in a high-grade farming locality and in another county. The location of the piggery in another county and among good farms brought objections at once from the neighbors. An injunction was sought against the operation of the plant but the Court held that no injunction could be issued until a nuisance had been created. Later, Court action was again invoked and on February 16, 1925, an injunction was granted by the Circuit Judge of the county in which the pig farm is situated. This injunction was based on testimony that nuisances had been caused by foul odors from the pigs and more especially from the disposal of refuse from the feeding floors. The Court decided that the neighbors had been damaged in the enjoyment of their homes and that the value of their properties had been decreased. The case is now before the Supreme Court of Michigan for final review. Awaiting the decision of the Supreme Court, the City continues to operate the plant with improved methods of disposal of the waste materials from the feeding floors.

The feeding plant proper occupies a tract of about 8 acres near the center of the farm and 700 or 800 ft. from the nearest road. The animals have a free run on this area. There are three feeding houses, each 33 by 100 ft., with concrete floors. These houses are divided longitudinally by a fence into two alleys, 16 by 100 ft. The collection trucks drive into these alleys and unload direct from the cans to the feeding floors before the hogs are admitted. After the garbage has been fully worked over by the pigs, the rejected refuse is thoroughly treated with lime. Formerly this refuse was hauled in dump wagons to various fields on the farm and placed in one-wagon load piles. In the spring these piles were spread and plowed under as a fertilizer. So much odor nuisance was created by this operation that it was decided to distribute the material over the land by means of manure spreaders as fast as it accumulated. This seemed to be a marked improvement and less odors were created. After the injunction was granted it was decided to institute further precautions and the material is now buried in shallow trenches. The trenches are excavated with a road grader, the refuse is deposited in layers about 4 in. deep and covered by hand labor with about 8 in. of earth. It is expected that the land thus treated will be ready for crop use in about a year. This method of refuse disposal has eliminated all odors except pig smells which are not often noticed on the surrounding roads.

The garbage cans are thoroughly washed in a machine constructed especially for that purpose in a building near the feeding houses. This building also contains a room with steam pipes in the floor for thawing garbage in the cans so as to avoid damage in removing frozen garbage, and a steam plant for heating and to furnish hot water for can washing and floor scrubbing.

Sleeping quarters for the hogs are provided in three buildings, 15 by 80 ft., two buildings, 15 by 100 ft., also about 10 old colony-houses about 6 by 8 ft. The new sleeping buildings are divided into pens about 15 by 8 ft. There has been some trouble from sweating of the hogs with consequent danger of pneumonia. Decided improvement, however, has been obtained by building wooden box ventilators, 4 by 6 in. in size, that extend from about 2 ft. above the floor through the roof. There is a ventilating shaft for each pen. The

sleeping buildings are in pairs, with the space between the buildings paved with concrete. The entire yard around the buildings is scraped once each week to remove droppings.

Pigs are bought at a weight of about 120 lb. and kept until they weigh about 250 lb. They are kept in quarantine for 21 days during which time they are inoculated against hog cholera and pneumonia. The number of animals averages about 1 200 in winter to about 2 000 in summer.

The garbage received at the farm is not weighed and no accurate figures are obtainable to show how much is received during a year, but for 1924 the quantity is estimated at 19 350 tons. This figure was obtained by multiplying the total number of cans collected by the average weight of the contents of the cans.

The cost of disposal for the eight months, from May 1 to December 31, 1923, is as follows:

Receipts:		
Sale of hogs	$20 211.60	
Farm produce	905.84	
Increase in value of hogs on hand	13 500.00	
Total	$34 617.44	$34 617.44
Expenses:		
Hogs purchased	$15 652.37	
Pump and repairs	193.18	
Taxes	86.50	
Interest at 6% on $15 000 (farm)	600.00	
Insurance	32.37	
Lime	331.00	
Labor	7 782.17	
Repairs to buildings	122.92	
Total	$24 800.51	
Interest and Depreciation:		
Farm implements	$22.90	
Buildings	716.05	
Total	$25 539.46	$25 539.46
Profit for 8 months		$ 9 077.98

It is unfortunate that records are not available to show the financial results of operating the piggery over a period longer than eight months. It is believed, however, that these figures are not exceptional and that the profits during 1924 and in 1925 up to the time that burial of the refuse from the feeding floors was begun, have been similar to those shown for the last eight months of 1923.

CALIFORNIA PRACTICE OF GARBAGE DISPOSAL BY HOG FEEDING

By W. T. Knowlton,* M. Am. Soc. C. E.

Although this paper is entitled "California Practice of Garbage Disposal by Hog Feeding", the data relate only to the City of Los Angeles. From July, 1915, to September, 1921, all garbage of the city was delivered to a reduction plant owned and operated by a private company. In 1915, this company made a 10-year contract to dispose of the garbage, but, in 1921, it cancelled its contract as it claimed it was losing $15 000 per month.

In September, 1921, the City entered into a 10-year contract to sell its garbage for hog feeding. The garbage is collected by the City and shipped on steel gondola cars to the hog ranch of the Fontana Farms, which is in San Bernardino County, 55 miles from the loading station in the city. The City receives $0.60 per ton for the garbage after loading on the cars.

Before leaving the city, the garbage is classified as to whether it comes from the hotels and restaurants or from residential districts. In the business district, garbage is collected every night, and in the apartment house districts and thickly populated residential districts collections are made three times per week. In the general residential districts, collections are made twice a week. The garbage collected from restaurants is used in the fattening of hogs, the other garbage being used for the smaller hogs which have not reached the fattening stage. Spur-tracks laid between the pens permit the garbage to be unloaded directly by cranes on to the pen floors, which are of concrete. To keep all areas where feeding takes place in a sanitary condition, all the spaces adjoining the railroad tracks and the drainage gutters are also paved with concrete.

The total area of the Fontana Farms is about 14 000 acres, of which, 320 acres are used for a hog ranch. Including the brooder group, there are about 44 000 hogs on the ranch, divided into different groups from the weaning stage to the fattening stage. The quantity of garbage fed to the hogs in the various groups depends on their development. After feeding, the waste is piled by plows and Fresno scrapers, and removed by wagons. This clean-up is then taken to a compost pit.

In 1921, an average of 216 tons of garbage was collected per day, the daily averages for the following years being 246 tons in 1922, 324 tons in 1923, 371 tons in 1924, and 371 tons in 1925. A part of this daily garbage is material that the hogs will not consume. In addition to this waste, the clean-up from the pastures and back pens is also collected. The total of this refuse is about 125 tons per day, of which 50 tons come from the feed floors, 40 tons from the back-pen clean-up (including therewith the hog manure), and 35 tons from the pasture clean-up.

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The compost pit consists of two concrete bins, each 250 ft. long and 12 ft. deep. The side walls of each bin are 56 ft. apart at the bottom and 75 ft. at the top. The concrete floor of each bin consists of alternate ridges and valleys equally spaced. The horizontal distance between each valley and ridge is 7 ft. and the vertical distance about 1 ft. Below each valley line is a concrete box-drain about 1 ft. square in cross-section, in which the drainage from the waste material is carried on a slope of 1 in 20 to an intercepting channel at the end of the bin. This intercepting channel discharges its flow into a sump from which the water is pumped into a sewer. A movable steel girder bridge, 75 ft. long, across the bin permits the waste material to be readily dumped into the compost pit, the bridge moving along rails laid at the top of the side of the pit.

When the feed-floor clean-up is dumped into the pit, it is covered with gypsum to prevent evaporation of the ammonia content. Peat was first used for such a cover, but later was abandoned, as it prevented moisture in the waste material from reaching the drains. Distillate is sprayed over the waste dump to prevent the breeding of flies. After being in the pit for four months, the material is removed by trucks, from an entrance at one end of the bin, and placed on a concrete floor, 120 ft. wide, that extends along the entire length of one side of the concrete bins, the floor being at the same level as the top of the bins. The clean-up from the pasture and back pens is spread on this floor to dry for a few days.

When placed in the compost pit, the waste material contains about 75% moisture, which is reduced to about 42% when it is removed to the drying floor. The compost product, together with the other clean-up after drying, is ground into a fine material which is hauled away to be used as a fertilizer in the orchards and vineyards of the Fontana property. An analysis of this ground-up product gave a water content of 5½ per cent.

The writer in interviewing those in charge of the hog ranch finds that they prefer to receive the garbage not more than a day old. Garbage is not wanted if it is more than 4 days old. The hogs are fed twice each day, the average daily meal being about 20 lb. per hog. To operate the hog ranch 3 trucks and 12 two-horse teams are required, together with a force of 110 men, mostly Mexicans.

THE DISPOSAL OF ORGANIC WASTE BY THE BECCARI SYSTEM AT SCARSDALE, NEW YORK

By Arthur Boniface,* Assoc. M. Am. Soc. C. E.

Every rapidly growing community, sooner or later, faces the problem of collecting and disposing of its household waste, in a sanitary, well-regulated, and economical way.

In 1922, the Village of Scarsdale, N. Y., having a population of 4 500, decided to provide for the collection and disposal of its waste by village forces. This decision was made with a view to correcting unsatisfactory conditions resulting from the then existing practice of doing such work by private contractors.

The village embraces about 7 sq. miles of territory and is almost wholly zoned for residential purposes, which has resulted in establishing a community of uniformly high-class homes. In March, 1923, a bond issue was authorized to provide funds for acquiring a suitable site and for the construction of a garbage disposal plant. Meanwhile, all the methods of garbage disposal were studied, particularly the system developed in Italy by Dr. Beccari.

The Beccari System first contemplated the reclamation of organic waste accumulated on farms, with a view not only to its salvage, but to an increase in its value and effectiveness as a fertilizer. Later, Dr. Beccari applied his system to the disposal of organic waste produced by municipalities, and met with complete success. From two cells built and operated in Italy in 1914, there are now in use, or being built, in that country, about 1 800 cells.

It may be a minor consideration, but in communities like Scarsdale, the fact that a Beccari plant lends itself to architectural treatment is important, as much of the objection to other systems of disposal can be overcome. Having in mind the character of development in Scarsdale, and the fact that a disposal plant anywhere within the village confines might create a hardship to adjacent property owners, and faced with the necessity of a plant of some kind, the Beccari System had a peculiar appeal to those responsible for the disposal of that community's waste. The Beccari System was therefore chosen and an eight-cell plant built.

The Scarsdale Plant

It was anticipated that the Scarsdale plant would take care of an ultimate population of 8 000. It might be stated that, from records carefully kept, the production has been found to exceed all estimates and experiences, averaging almost 1 lb. per capita.

The plant commenced operation on December 3, 1923, and is now (January, 1926) disposing of the waste accumulated from about 6 000 people. It consists of four units, or eight cells, each cell having a capacity of 25 cu. yd.

* Village Engr., Scarsdale, N. Y.

Extension sheds were constructed on the sides and the tops of the cells, which, in addition to providing storage for the humus, furnish housing for the trucks and equipment. The charging is done through hatches on top of the cells and the discharging through doors built on the sides.

Each cell has a double floor, the bottom one draining to the sump. The upper floor consists of gratings, the openings in which equal 30% of the total floor area. At the four interior corners of the cell, ducts are built with openings at the height of the horizontal baffles or fillets which extend around the interior walls of the cell about 2 ft. apart. The object of these baffles is to assist in distributing the air through the mass, the air being admitted, in the first instance, through two port holes between the upper and the lower floors. Connected by a duct through the roof of the cell is a turret or tower in which are constructed five trays with openings at alternate ends. By this means, the escaping gases generated in the mass below, pass over each shelf, successively, before discharging into the open air.

Spread over the trays is an argillaceous material which, in turn, is covered with crystals of sulfate of iron. Its function is twofold: It fixes the products of the gases rising from the fermenting mass below and absorbs unpleasant odors that might otherwise escape and become a nuisance. The drip from the trays trickles back into the mass and undoubtedly assists in contributing to the high percentage of nitrogen and other chemical elements found in the residue after the cycle is completed. The cells function through the bacterial action of the micro-organisms developed in the fermenting mass.

Operation of Scarsdale Plant

The Scarsdale plant commenced operation by charging Cell No. 3 with about 45 cu. yd. of raw garbage. In all, about 17 cu. yd. in a period of 5 days were placed in this cell for its initial charge. On the third day after beginning operation, the characteristic drip became evident, and two days later this had increased very perceptibly. Also, on raising the trap-door above the cell, considerable heat could be felt, and vapor arose from the mass, showing that the first stage of putrid fermentation was in full progress.

At about the end of the third week the numerous fluidizing colonies prevalent in the early stages were observed to disappear, and a mould or fungus began to put in an appearance. There was no apparent diminution of heat even up to the time of discharging the mass which took place 45 days after the closing of the cell. The residue or humus from this first cell, and that of each subsequent cell as it was discharged, was stored in the shed adjacent, with the object of drying it and afterward grinding it for commercial purposes.

The humus as stored in a pile did not dry out as anticipated, and, further, at the foot of the pile, combined with the drip which continued, an appreciable hydro-carbon deposit was observed. It was thought that a longer period in the cell might reduce the moisture content of the humus, and, therefore, experiments were made leaving the mass undisturbed for periods varying from 50 to 65 days, but even with the longest period, no particular difference was noticed in the character of the humus produced, or its moisture content.

Efforts were made to dry the product in the open air on a platform erected just outside the plant, but this expedient was unsuccessful, and it became apparent that a properly designed drying shed must be constructed. Later, a shed, 100 ft. long and 12 ft. wide, was built on a farm in an adjoining township. It was constructed on sloping ground and permitted an arrangement, without great expense, whereby a lower floor accommodated the grinder. The humus from a cell was first dumped at one end of the shed and spread to a depth of about 2 ft.; then, at intervals of 2 days, it was turned over and gradually spread toward the other end of the shed above the grinder. At the end of 10 days it was sufficiently dry to be passed through the grinder, and sacked, after which it was sold as fertilizer. It was thus demonstrated that the humus could be made commercially valuable by natural processes. In using this means, however, there is danger, in built-up communities, of creating objections due to odors.

Control of Odors

There is another condition which must be met, namely, the control of odors during the discharge of the cells. It should be borne in mind that as the organic matter breaks down, the mass contracts to about two-thirds of its original volume. It, therefore, attains great density and compactness. An interesting fact, to which attention should be directed, is that this contraction is not accompanied by as great a reduction in the moisture content as might be expected. The effect of all this is the retention in the mass of quantities of ammoniacal and nitrogenous gases, which are naturally liberated when the cell is discharged. Under certain atmospheric conditions this is likely to be a serious matter.

In addition to the objectionable odor, the mass is in no condition to pass through a grinder, as by analysis it was found to contain about 45% moisture. The problem, therefore, is to break up the mass in the cell and induce the passage, if possible, of larger quantities of air. With this in mind, latticed wood cones, about 2 ft. square at the base and 4 ft. high, were built, the spacing of the bars being arranged to give each cone about 5½ sq. ft. of free openings.

The object was to increase by 200% the original free area provided by the floor gratings, and, therefore, seven of the cones have been placed in one of the cells to determine the effect on the mass. In addition to this experiment an attempt at drying the humus in the cell is now (1926) under way. For this purpose a fan, with a 25-h.p. motor, capable of handling 2 000 cu. ft. of air per min., has been installed. On the suction side of the fan, heating coils, connected with a 15-h.p. steam boiler, have been placed.

It is proposed to commence operation of this blower at the conclusion of the fermentation cycle, and maintain the temperature of the air passing through the mass approximately equal to that which the mass under normal conditions would develop, and to continue such operation until the mass is dry. This experiment will be watched closely and all the effects noted. Already the insertion of the wood cones in the cell has resulted in advancing the various phases of the fermentation cycle by several days.

Conclusions

As this paper must be considered more in the light of a progress report, definite conclusions will not be stated. The results to date, however, can be summarized as follows:

The plant was inexpensive to construct on account of its simplicity, and its operation has required only labor of the ordinary class.

Except for $6 for the purchase of the sulfate of iron for charging the turrets, the plant has cost nothing for maintenance.

The analysis of the humus shows the following average content:

	Percentage.
Nitrogen	3.0
Phosphoric acid	3.5
Potash	0.6
Moisture	45.0

Experiments with the humus in vegetable gardens and in flower beds has definitely shown its agricultural value.

The arrangement for absorbing odors from the cells during the fermentation process is adequate and entirely successful.

Some means must be devised to reduce the moisture content of the humus while it is still in the cell in order to avoid objectionable odors during its discharge and to facilitate its preparation for the market.

Although the moist humus, as taken from the cell, emits a characteristic odor, no trace of odor of any kind can be detected when the humus has dried.

It has been conclusively demonstrated that the system is peculiarly adaptable to communities like Scarsdale, and other high-class residential developments.

Also, with a properly designed drying unit, and perhaps an air washer, there is no limit to its applicability for the disposal of organic wastes.

In conclusion, there is no doubt that a new means of disposal of organic waste has been devised; that it is one of the most scientific and sanitary methods yet invented; and that, with the perfection of one or two minor details of operation, it will prove of great value to those charged with the responsibility of the disposal of community waste in a safe and economical manner.

HIGH-TEMPERATURE INCINERATION AT TORONTO, ONTARIO, CANADA

By J. A. Burnett,* Esq.

This paper is intended to outline briefly the principles of refuse disposal at Toronto, Ont., Canada, by the method of the high-temperature destructor.

Prior to the re-organization of the civic departments in 1913, most of the city refuse was dumped, the remainder representing about 75 tons per day, was destroyed by a low-temperature natural-draft incinerator of somewhat antique design, which was built about 1890.

Both methods were entirely unsatisfactory from a sanitary viewpoint. The old incinerator was rebuilt and modified to improve operating conditions, and after careful studies and surveys, the Street Cleaning Department, jointly with the Medical Health Department, finally recommended disposal by a high-temperature destructor. This recommendation was endorsed by the City Council, which authorized an expenditure of $1 000 000 for the refuse disposal program.

Early in 1917, a plant with a guaranteed capacity of 180 tons in 24 hours, was completed and placed in operation. This plant, which was built to the Department's specifications, consists of three, high-temperature, "Sterling", 4-cell furnace units, with combustion chambers, air heaters or regenerators, connecting flues, etc., and all appurtenances, including a radial brick chimney. The performance records of this plant have been very satisfactory during the eight years of its operation. Approximately 50 000 tons of mixed refuse is disposed of annually, which is a furnace capacity of 4½ tons per hour (the guaranteed capacity being 2½ tons per furnace-hour).

The plant is in close proximity to at least five public institutions and is quite near a good class residential district. It is situated on low-lying ground with banks on three sides; the vehicles delivering material enter and leave the plant over bridges almost level. There are no complaints on record with respect to the operation of the plant.

During 1923, an additional destructor plant was recommended by the Department, to replace the old crematory, and also to provide for the disposal of considerable refuse that was being dumped. Specifications were prepared for a system similar to that of the former destructor, with the improvements incorporated that were found to be of benefit in the experience of operation and maintenance. The new plant, which is known as the Wellington Destructor, was placed in operation early in 1925, and finally accepted on May 1, 1925. It consists of four high-temperature, "Sterling", 4-cell furnace units, with combustion chambers, air heaters, flues, etc., and all appurtenances, including two

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radial brick chimneys, complete with self-supporting fire-brick lining, which extends the full height of the chimneys. Each chimney is 175 ft. high, with an internal diameter of 90 in. The guaranteed capacity of this plant is 400 tons per 24 hours.

The building of each plant was designed by the City Architect's Department, which also supervised the construction. The work was done by local contractors, but all engineering and supervision of the plant equipment was under the jurisdiction of the Engineering Division, Department of Street Cleaning. Both plants are of fire-proof construction throughout. The cost of the former, or Don Plant, was $225 000, and that of the Wellington Plant, $550 000, exclusive of land in both cases.

The furnaces are of the continuous-grate regenerative type, the cells being divided by low-set air-cooled castings which render it possible to fire and clean the grates separately. There are no moving parts inside the furnaces; each furnace unit has four cells, and a charging container for each cell; common combustion chamber, air heater, and by-pass flue. The material from the charging container drops on a drying hearth at the back of the grates, afterward being moved forward and spread uniformly on the grates, where incineration takes place. The gases pass over the grates into the combustion or settling chamber. At the back of this chamber a fire-brick arch splits the gases, a part of which pass into the regenerating chamber, and through the air-heater tubes, the remainder entering the by-pass flue, thence through the connecting or main flues into the chimney. The cool gases after passing through the air heater also discharge into the main flue.

Substantial steel buckstays are set at frequent intervals, securely tied top and bottom with rods to resist expansion stresses. All skewbacks are supported by heavy steel angles extending the full length of the various arches. The fire-brick lining of the furnaces, combustion chambers, and flues, is not less than 9 in. thick, and the outside walls are $1\frac{1}{2}$ brick thick, except the connecting and main flues, which are only 1 brick in thickness. An air space of at least $\frac{1}{2}$ in. between the lining and walls is maintained throughout.

The general arrangement of the flues is such that any desired battery of three units may be operated with either chimney, so that the plant may be operated at not less than 75% capacity at all times. The dimensions of the combustion chambers are such as to permit a rolling action of the gases. The approximate flue velocity is 20 to 25 ft. per sec.

The quality of the material delivered to the plants, as in every municipality, varies to some extent. The refuse delivered to the Wellington Plant is more uniform than that hauled to the Don Plant, which on certain days contains a greater proportion of rubbish. However, the average material for both plants is about 55% moisture, 34% combustible, and 11% residue. Burning this grade of material, an average combustion chamber temperature of 1 600 to 1 800° Fahr. is maintained at the rate of 4 to 5 tons per furnace-hour. The refuse consists of a mixture of rubbish and garbage, the approximate proportions being 60% garbage and 40% rubbish, by weight; no ashes or incombustible waste is hauled to the plants.

The rate of burning at the new plant is greater than that at the old one, on account of the increased width of the drying hearth, which renders the total grate area effective for burning, although the Don furnaces have an equal grate area, 25% of which at least functions as a drying hearth. At both plants, all material entering as well as the residue is weighed. The reduction by weight is 89 per cent.

Electric motor-driven blowers furnish the forced draft. Those installed at the Wellington Plant are equipped with dual inlets. When the furnaces are burning, the air is drawn from the charging floor level, but when the grates are being cleaned, the smoke is drawn from the ash-run and returned into the furnace.

The fans at the Don Plant furnish 7 500 cu. ft. per min. at a static pressure of 6-in. water gauge, whereas those at the Wellington Plant furnish 10 000 cu. ft. per min. at a static pressure of 4-in. water gauge. The estimated quantity of air required is 416 per lb. of refuse.

The pre-heated forced draft is considered to be a marked improvement in the development of the high-temperature destructor. It is understood that the Don Destructor was the first plant in American practice in which the gases at final combustion-chamber temperature were passed through the air-heater tubes. The method, however, has been successful to the extent that, after seven years of continuous operation, the bottom tube plates of one heater only have been replaced. Cast-iron tubes, 5 in. by $\frac{5}{16}$ in. thick, were installed for the Don heaters. Seamless steel tubes were furnished for the Wellington Plant, which have the advantage of transferring heat more rapidly, and also of reducing materially the dead load on the bottom tube plates. The durability of the steel tubes as against those of cast iron, due to the action of the gases, is problematical.

Pre-heated forced draft to a final temperature of 300° Fahr. (obtained normally), is a great factor in efficient burning. At this final temperature, the theoretical heat value put into the air, which is equivalent to 40 to 50 lb. of coal per ton of refuse, is claimed by some authorities to be a great advantage. However, from a practical operating viewpoint, the air is more efficiently distributed throughout the fuel bed on the grates, than it would be if coal were supplied to furnish the same heat. Also the increase in volume, which is approximately $1\frac{1}{2}$ times the volume at initial temperature, aids combustion by a better distribution throughout the refuse for the supply of oxygen, thereby requiring less excess air than would be the case without pre-heating. The capacity when operating under natural draft has been found to be 60% of that under forced draft conditions.

The furnaces of both plants are charged by hand, each charging container having a capacity of approximately $\frac{3}{4}$ cu. yd. With the system of light charges at frequent intervals, the best burning results are obtained, namely, incandescent fuel bed, high rate of burning, and uniform temperatures.

Grates are normally cleaned three times each shift of 8 hours. The charging container and furnace doors are operated by compressed air rams.

The labor requirements are as follows: 1 foreman of plant; 1 leading stoker per unit for each shift; 3 charging men; 2 stokers; and 1 ash-run man. Table 4 shows briefly the statistics for 1925.

TABLE 4.—OPERATING STATISTICS OF INCINERATING PLANTS, TORONTO, ONT., CANADA, FOR 1925.

Plant.	Total experimental laboratory report, etc.	Loads, in pounds.	Tonnage.	Man-hours.	Man-hours per ton.	Residue, in tons.	Percentage of residue.
Don.........	$75 648	44 940	50 159.708	76 746	1.55	5 265	10.5
Wellington...	70 448	53 286	63 351.430	83 806	1.33	6 983	11.

The total cost of operation including labor, repairs, materials, and supplies, less carrying charges, for the Don Destructor is $1.50 per ton and for the Wellington Destructor, $1.11 per ton. No repairs or renewals were made for the Wellington Plant during 1925. Daily records of the complete operation of these plants have been made, dating back to July 1, 1917, for the Don Plant, and to January 1, 1925, for the Wellington Plant. Combustion-chamber temperatures are recorded continuously by electric recording pyrometers.

THE COBWELL SYSTEM OF GARBAGE REDUCTION AND SOME PHASES OF ITS OPERATION AT ROCHESTER, NEW YORK

By John V. Lewis,* Esq.

Historical

The idea of the "Cobwell System" of garbage reduction was conceived by Mr. Raymond Wells in 1913, who spent the next two years in the development of his system, in collaboration with the C. O. Bartlett and Snow Company, of Cleveland, Ohio. As a result of their experiments and endeavors, the first practical Cobwell Plant commenced operation at San Francisco, Calif., early in 1915. This was a small 2-unit plant, with a capacity of 10 tons per day, for handling the garbage of the Panama-Pacific International Exposition.

About the middle of March, 1915, the Pacific Reduction Company opened the first large Cobwell Plant at Los Angeles, Calif. This plant contained 48 Cobwell units with a total daily capacity of 240 tons, and its operation was continued over a period of about six years, the plant at times handling as much as 300 tons per day.

Subsequently, four Cobwell reducers were installed at the plant of the New Bedford Extractor Company, New Bedford, Mass., and their use continued until the plant was destroyed by fire in June, 1923.

The largest Cobwell Plant installed to date was placed in operation July, 1917, by the Metropolitan By-Products Company on Lower Staten Island, Borough of Richmond, New York, N. Y. This huge plant was provided with 196 Cobwell units and had a capacity of about 1 300 tons per day. It was constructed to supplant the old Barren Island Plant. However, its operation was discontinued after a period of about fifteen months and has never been resumed.

During the next four years only two Cobwell Plants were built, one at Syracuse, N. Y., and the other at Rochester, N. Y. The Syracuse plant was a 12-unit installation, capable of handling 60 tons per day. In May, 1923, however, preliminary dehydrators were added, thereby changing the process to the modified Cobwell System and increasing its original capacity to 85 tons per day.

The Rochester Plant consists of 36 Cobwell units and has a maximum rated capacity of 180 tons per day. It has now been in continuous operation for a period of more than four years, in strict adherence to the original principles of the Cobwell System. Excluding the plant of the Pacific Reduction Company at Los Angeles, the Rochester Plant has to its credit the longest period of unmodified operation of the several plants thus far constructed and is now the only straight Cobwell Plant in use.

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During 1925, operation of a modified Cobwell Plant was commenced at Schenectady, N. Y. This plant contains only four Cobwell reducers but, with the dehydrator equipment, it is capable of handling 90 tons per 24-hour day. A plant of similar type, with a daily capacity of about 400 tons, has recently been proposed for Cleveland, Ohio.

This summary and Table 5 outline briefly the history of the development and installation of the "Cobwell and Modified Cobwell Systems" over the short period of ten years. Compared with the older systems of garbage reduction still in use after a period of forty years, it represents a distinct advance from the sanitary standpoint. Engineers may differ as to its economic operation or advantages, in comparison with other sanitary methods of garbage disposal, but this fact remains: The plants which are both in and out of operation stand as milestones along the path of progress in the field of municipal sanitation and the "Cobwell" or a similar type of system will supplant the older processes in those cities which have always looked with favor on reduction as a means of garbage disposal and have used it with more or less apparent success.

The Cobwell System

The most important part of the Cobwell System equipment is the so-called "reducer". This machine is very simple in design, consisting of a flat cylindrical steel shell about 10 ft. in diameter and 4 ft. high. A steam jacket is provided under the bottom and around the lower two-thirds of the circumference of the shell. A set of cast-steel plow arms, mounted on a vertical shaft with top and bottom bearings at the center of shell, are rotated close to the inner bottom sheet of the reducer. These arms slide under the mass of garbage, imparting to it an undulatory motion.

The mechanism for driving the plow-arm shaft at 10 rev. per min. is mounted on the flat top of the reducer. A charging door; a peep-glass for observing the action within the reducer; and an outlet for water and solvent vapors are spaced about on this same top. The discharge door is set into the reducer shell at the low point of its circumference, and several perforated drain-boxes project upward into the bottom of the reducer. These boxes serve not only as outlets for removing the grease extracted from the garbage, but also as inlets for the volatile solvent and live steam introduced at various periods in the reduction cycle. Each reducer is equipped with an individual steam trap and the necessary steam, solvent, and drain-valves, drain sight glass, and piping, and is supported on a heavy structural iron frame.

In other respects, the layout of a Cobwell plant does not differ greatly from that of the older systems. The usual types of conveying equipment for handling the green garbage and the rough and finished tankage are provided. There are electric prime movers and power-transmission equipment; surface condensers and condensate, solvent, and grease pumps; and steam stills, and the various separator and storage tanks for water, solvent, and grease. There is a mill room, equipped with crushing and screening machinery, for converting the rough tankage into a granular fertilizer base.

TABLE 5.—HISTORICAL DATA ON GARBAGE DISPOSAL PLANTS UTILIZING THE COBWELL SYSTEM OF REDUCTION.

Plant located at:	Type.	No. units.	Rated capacity, in tons per day.	Operated by:	Operation begun:	Period of operation.	Remarks:
San Francisco, Calif.....	Straight	2	10	C. O. Bartlett & Snow Co....	1915	10 months	Panama-Pacific Exposition.
Los Angeles, Calif.......	Straight	48	240	Pacific Reduction Co.........	1915	6 years	Operation discontinued in 1921.
New Bedford, Mass......	Straight Modified	4 4*	20 ...	New Bedford Extractor Co.. City of New Bedford	1915	8 years	Destroyed by fire in 1923.
New York, N. Y..........	Straight	199	1 300	Metropolitan By-products Co	1917	15 months	Operation discontinued in 1918.
Syracuse, N. Y..........	Straight Modified	12 12*	60 85	City of Syracuse............. Cobwell Reduction Co., Inc..	1921 1923	5 years	In continuous operation.
Rochester, N. Y.........	Straight	36	180	City of Rochester............	1921	4 years	In continuous operation.
Schenectady, N. Y.......	Modified	4*	90	City of Schenectady..........	1925	9 months	In continuous operation.
Cleveland, Ohio..........	Modified	....	400	City of Cleveland	....	....	Proposed installation.

* With preliminary dehydrators.

The straight Cobwell process in itself is carried on to its conclusion as a single continuous operation in the closed reducer from which odors do not escape. Furthermore, after a reducer has been charged, the several operations utilized in the older processes and involving the use of hand labor and the movement of the garbage to and from different kinds of equipment are not required. The reduction cycle consists of five steps as follows:

(1) Loading or charging of the garbage into the reducer.

(2) Dehydration or cooking, for the removal of free and combined moisture.

(3) Washing or extracting of the grease and oils.

(4) Drying or steaming of the tankage.

(5) Dumping or discharging of the rough tankage from the reducer.

The complete reduction cycle consumes from 18 to 24 hours, dependent on the character and condition of the garbage. The so-called cooking period requires from 14 to 17 hours and the washing period, with three washes, from 3 to 4 hours. Steaming of the tankage ordinarily occupies 1 hour. The loading and dumping periods are comparatively short, of 10 to 20 min. duration only.

Perhaps the most interesting feature of the whole process is that of the temperature produced during the reduction of the garbage mass. Entirely submerged in a bath of petroleum naptha, which acts not only as a heat transfer medium, but also a preventive of bacterial growth and hydrolysis, the garbage gives up its moisture content and disintegrates at a temperature of about 208° Fahr. This low temperature is not exceeded at any time in the reduction cycle and therein lies the reason for the non-production of obnoxious odors. The so-called "digestion" and digestion odors common to the older systems are not produced.

Aside from the important sanitary factors, the Cobwell System provides a higher recovery and larger quantity of valuable by-products than the older systems and a superior quality of the tankage as well.

Garbage Collection and Reduction at Rochester, N. Y.

From the best available information, it appears that the collection and disposal of the garbage of the city, as a municipal function, was undertaken about 1880. During the next thirty-seven years, this work was done mostly under contract with private collectors and contractors. In the first twenty-six years of this period, the collected material was hauled outside the city limits and disposed of by dumping or burying. When, in 1904, due to numerous complaints, it became evident that some other and more sanitary method of disposal must be adopted by the City, Edwin A. Fisher, M. Am. Soc. C. E., then City Engineer, was empowered to make a special survey and study of the situation, with recommendations for a better system of collection and disposal than was then in use. Mr. Fisher completed his report in September, 1906, and, as a result of his recommendations, the City awarded a 10-year contract for the collection and disposal of garbage, effective from January 1, 1907, to the Genesee Reduction Company. This Company proceeded to acquire a site on the west bank of the Genesee River between the Upper and

Lower Falls. The location chosen was within 500 ft. of State Street, an important business street, and within ¾ mile of the geographical and business center of the city. The narrow portion of the river bank at this point, on which the plant was built, lies about 150 ft. below the level of the city streets bordering on the river gorge and adjacent thereto. It is interesting to note that this same site is also occupied by the new "Cobwell System" plant which was erected in 1920-21. From the standpoint of the collection and haulage of garbage, it is an excellent location but, at the same time, a critical one with respect to possible nuisance caused by obnoxious odors which are inherent with the older systems of garbage reduction.

The Genesee Reduction Company began the operation of its plant early in June, 1907, using a modified Arnold, or so-called "Beaston", Process. At the conclusion of the 10-year period, the City purchased the collection equipment and the disposal plant from the contractor and has proceeded to carry on its own garbage collection and disposal systems since that time under the direction of the Department of Public Works. The modified Arnold Plant was operated continuously up to the date of the opening of the Cobwell Plant on October 15, 1921. Thereafter its use was discontinued except to utilize the digesters for grease-storage purposes and a few of the buildings for a repair shop and stock and dry storage rooms supplemental to the new plant.

The speaker can only surmise the conditions and reasons which brought about the adoption of the new system of disposal. Although formal complaints against the operation of the old plant might have been few, there is evidence that verbal objections were many and that considerable money and effort were expended in an attempt to overcome the obnoxious odors and conditions attendant on its operation. Furthermore, the original equipment had reached the point where repairs were becoming more and more frequent and costly, and the end of its useful life was in sight.

In 1912-13, the City had constructed and commenced the operation of a Rubbish Salvage and Disposal Plant adjacent to the Reduction Plant. It originally contained a single DeCarie incinerator, but the capacity having been exceeded, the City proceeded in 1916-17 to install two new units, each with a capacity for burning 50 tons of rubbish per 24 hours. In addition to the salvage of saleable materials, the plant generated a considerable quantity of saturated steam which was purchased by the Rochester Railway and Light Company and delivered into its commercial heating mains. On the beginning of operation of the Cobwell Plant, the City diverted the supply of steam there, making it auxiliary to the supply of high temperature steam furnished to the new plant by the Power Company. This procedure has served to reduce the actual cost of the total quantity of steam required for operation of the new garbage plant in that each pound produced at the Rubbish Disposal Plant represents a saving of an equal amount which must be purchased in the absence of such incineration units and waste heat boilers or failure to operate them.

The Cobwell Plant and Some Phases of Its Operation

As has been heretofore mentioned, the new plant was opened in October, 1921. When, at the beginning of 1924, Harold W. Baker, Assoc. M. Am. Soc.

C. E., became Commissioner of Public Works, and the speaker was appointed Sanitary Engineer in charge of the operation of all refuse disposal plants, the Cobwell Reduction Plant was found to be badly in need of repairs. Its condition, after only 27 months of operation, was such as to cause excessive consumption of steam, solvent, and condenser-cooling water and to endanger the safety of not only the plant and its personnel, but of the adjoining properties as well. Furthermore, records of operation and data, particularly with reference to the quantity of garbage handled and the quantity and quality of by-products produced, were very meager and unsatisfactory. No means had been provided at either the old or the new plant for weighing the quantity of garbage delivered daily for treatment, and a suitable platform scale installation was one of the first additions under the new régime. Replacement parts were procured and the repair work prosecuted as rapidly as possible. A suitable system of record keeping was inaugurated and the operation of vital plant equipment checked by means of the installation of steam flow meters and pressure, vacuum, and temperature instruments of the various indicating, recording, and integrating types. Almost immediately, a new spirit of co-operation and endeavor began and within the short period of six months, the equipment had been restored and the plant was again operating as its builders intended it to perform. Thereafter, everything possible was done to achieve satisfactory results in the daily routine use of the process and, at the same time, reduce the operating costs at the plant. It is beyond the scope of this paper to discuss in detail the experiments and developments carried out during the past two years, a study which has furnished much of interest from the chemical, mechanical, and economic standpoint. The speaker ventures, however, to present for the information of interested sanitary engineers and municipal officials, a summary of cost of operation during this period. These data are given in Tables 6 to 9, inclusive.

TABLE 6.—COST OF GARBAGE DISPOSAL BY COBWELL PROCESS AT ROCHESTER, NEW YORK. CAPITAL CHARGES.

Item.	Cost.	Interest and depreciation.*
Land	$47 140	$1 885
Buildings	138 090	82 130
Equipment	568 960	
Total capital charge		$84 015
Capital cost per ton of garbage treated		1924†..33 200 tons..$2.50 per ton. 1925 ..34 700 tons..$2.40 per ton.

* Interest charges are variable from 4 to 5 per cent. Depreciation charges are estimated at 2% on buildings and 8% on equipment.

† The quantity of garbage treated during the first three months of the 1924 period is estimated.

Table 6 sets forth the capital charges on the land, buildings, and equipment which constitute the Cobwell Plant. These charges have been reduced to a tonnage basis for the years 1924 and 1925. In Table 7, the several major

items which make up the operating costs of the plant are indicated over the same period of time and for the respective yearly tonnage. Table 8 presents the quantity of saleable materials produced, together with the revenues obtained therefrom in 1924 and 1925, and an estimate of the probable revenues for 1926. In Table 9, the total cost for the collection and disposal of garbage in 1924 and 1925 is summarized. These tables are concise, and the speaker desires to explain and amplify certain of the figures given therein.

TABLE 7.—COST OF GARBAGE DISPOSAL BY COBWELL PROCESS AT ROCHESTER, NEW YORK. OPERATING COSTS.

Item.	Period.			
	1924		1925.	
	Amount.	Percentage of total.	Amount.	Percentage of total.
Salaries and labor	$74 225	25.0	$72 285	32.0
Steam	84 730	29.0	67 050	29.0
Power and light	9 230	3.0	9 890	4.5
Solvent	43 670	15.0	23 550	10.0
Equipment and repairs*	40 115	14.0	20 640	9.0
Supplies and miscellaneous	31 050	11.0	25 335	11.0
Royalties	9 125	3.0	10 100	4.5
Total operating costs	$292 175	100.0	$228 850	100.0
Operating cost per ton of garbage treated			1924..$8.80 1925..$6.60	

* Includes cost of all new equipment and extraordinary repairs.

In the matter of capital charges, that for disposal is about the same in both 1924 and 1925, due to the slight increase in the tonnage treated in the latter year. No capital charges for collection are indicated, because the actual depreciation on the present equipment has been almost wholly absorbed. Such capital charges as were justified have been included as an arbitrary part of the collection operating costs during 1924 and 1925. In the operating costs for disposal, the largest items are those for labor, steam, solvent, equipment and repairs, and supplies. It will be noted that the cost for both steam and solvent, in 1925, was $18 000 to $20 000 less than that for 1924. The reduction in each case was due, largely, to the repairs made and the methods of operation introduced during 1924. At the same time, there was a substantial increase in the quantity of steam produced during 1925, at the Rubbish Disposal Plant. The cost of all extraordinary repairs that were made and of the new equipment installed during the two years, a good part of which might more properly be termed a capital charge, is included in the operating costs for the respective years. Altogether, about $65 000 has been expended for repairs and new and improved equipment during the two-year period. The low operating cost for collection of garbage is due, principally, to the central location of the disposal plant with respect to collection routes and to the kind

of labor and equipment which can be utilized under such circumstances. The average length of haul does not exceed 3 miles, and horse-drawn equipment is used almost exclusively.

TABLE 8.—COST OF GARBAGE DISPOSAL BY COBWELL PROCESS AT ROCHESTER, NEW YORK. REVENUES.

Item.	Period.		
	1924.	1925.	1926.
Quantity of grease produced, total pounds......	2 189 900	2 042 900	2 300 000*
Garbage treated, in pounds per ton......	66.0	58.9	65.0*
Grease revenue:			
Contract price per 100 lb......	$5.40†	$6.25†	$7.00†
Average net price per 100 lb......	$5.29	$6.00	$6.90*
Revenue per ton of garbage treated......	$3.50	$3.55	$4.50*
Quantity of tankage produced, total tons......	6 050	6 500	6 750
Garbage treated, in pounds per ton......	365	375	380*
Tankage revenue:			
Contract price per ton......	$8.10‡	$8.10‡	$8.10*
Average net price per ton......	$8.18	$7.90	
Revenue per ton of garbage treated......	$1.50	$1.50	$1.50*
Total revenue per ton of garbage treated....	$5.00	$5.05	$6.00*

* Estimated.

† Based on 97% saponifiable oils.

‡ Based on ammonia, 3.45%, $2.02 per unit; bone phosphate lime, 4.70%, $0.10 per unit; and potash, 1.10%, $0.60 per unit.

TABLE 9.—SUMMARY OF COST OF GARBAGE COLLECTION AND DISPOSAL AT ROCHESTER, NEW YORK.

Item.	1924	1925
COST OF COLLECTION: *		
Capital charge......	$0.00	$0.00
Operation......	3.35	3.45
Total......	$3.35	$3.45
COST OF DISPOSAL: *		
Capital charge......	$2.50	$2.40
Operation......	8.80	6.60
Total......	$11.30	$9.00
COST OF COLLECTION AND DISPOSAL: *		
Total......	$14.65	$12.45
Revenues......	5.00	5.05
Net......	$8.65	$7.40

* Per ton of garbage.

The revenues obtained from the sale of grease and tankage show that there was a fair market for these materials during 1924 and 1925. There are indi-

cations that the same conditions will continue through 1926 and perhaps for some time to come.

The figures given in the summary for 1925 are considered representative of those costs which may reasonably be expected at Rochester with the proper operation of the present collection and disposal systems. Deducting the revenues from the total cost for collection and disposal of garbage, the net cost per ton amounted to $7.40. If the cost of collection and the capital charges against the disposal plant are omitted, the operating cost for disposal alone is $1.55 per ton of garbage treated. There is every indication that this cost will be reduced during 1926 to $0.75 per ton, mainly through closer control of the process and the installation of new equipment.

In conclusion, the speaker can only say that the operation of the Cobwell Process during 1924 and 1925 has shown it to be a satisfactory means of garbage disposal, particularly from the sanitary standpoint. The desired results have been accomplished at a cost which is neither excessive nor prohibitive under the local conditions which are peculiar to Rochester and with a definite advantage over the methods which were previously used there.

THE NEW YORK STATE BARGE CANAL AND ITS OPERATION*

By Roy G. Finch,† M. Am. Soc. C. E.

During 1925 the New York State Barge Canal and its operation has been discussed at considerable length throughout the State of New York. This has been due in part at least to the functioning of a special legislative commission created for the purpose of ascertaining why the canal has not been handling a greater tonnage and of determining what could be done to stimulate traffic. The effectiveness and economic feasibility of this type of waterway is now being questioned, although prior to its construction in 1903 it was pronounced by engineering experts of recognized ability as the proper type of undertaking to afford a superior waterway connection between the Great Lakes and the seaboard. The writer welcomes this opportunity of presenting the facts with relation to this waterway system.

History of New York State Canals

In order to visualize properly present-day conditions it may be helpful to retrace briefly the history of the canals in New York State. The first waterway improvements were made by a private company chartered in 1792. By 1798 the natural streams of the State had been improved to facilitate traffic to a considerable extent, but it was not until 1817 that the State actually undertook the construction of the Erie Canal which was opened in 1825. The channel was 4 ft. deep and 28 ft. wide and floated boats carrying 30 tons. After the building of this canal the City of New York grew at an extraordinary pace and soon displaced Philadelphia as the Nation's chief seaport. So marked was the success of the Erie Canal that a veritable frenzy of canal building spread over the whole country, manifesting itself in New York State by the building of several lateral canals, six within the first decade after the Erie was completed, and four more within the next four years. To meet the constantly growing demands of traffic, the Erie and its main branches were enlarged from time to time. In 1862 the Erie Canal had a depth of 7 ft. and was capable of floating boats carrying 240 tons, a large increase compared with the first boats of 30-ton capacity on the original canals. By 1883, the year in which it was created a free canal by the abolition of tolls, the Erie Canal had earned a net surplus of nearly $43 000 000 in excess of its original cost plus the expenses of enlargements, maintenance, and operation.

Note.—Written discussion on this paper will be closed in January, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

* Presented at the meeting of the Waterways Division, New York, N. Y., January 21, 1926.

† State Engr. and Surv., Albany, N. Y.

In 1903, nearly ninety years after the starting of construction on the original Erie Canal, the people of New York State decided again to enlarge the canals by the building of what has been generally known as the "Barge Canal." This consists of the Erie Canal and the three principal branches of the canal system—the Champlain, the Oswego, and the Cayuga and Seneca Canals. The Erie is about 340 miles long and reaches across the State from Troy, on the Hudson River, to Tonawanda and Buffalo, on the Niagara River. The Champlain Canal runs north near the easterly boundary of the State from Troy to Whitehall, the southern end of Lake Champlain, a distance of 63 miles; the Oswego extends from a point on the Erie Canal near Syracuse to Oswego on Lake Ontario, a distance of 24 miles; and the Cayuga and Seneca Canal leaves the Erie west of Syracuse and runs southward, connecting with Cayuga and Seneca Lakes, a distance of 27 miles. Including the Hudson River and the lakes connected with the canals at various points and actually forming a part of the system, the total length of the New York State Barge Canal System is about 800 miles, of which about 70% is either in the beds of natural streams or in lakes.

Physical Features of Barge Canal

From tide-water level at Troy, the Erie Canal rises through a series of locks in the Mohawk Valley to Elevation 420 at the summit level at Rome and descends to Elevation 363 at the junction with the Oswego Canal and then rises to Elevation 566 at the Niagara River, a total ascent of 623 ft. and a descent of 57 ft. The Champlain Canal ascends from tide-water at Troy to Elevation 140 at the summit level at Fort Edward and then descends to Elevation 97 at the entrance to Lake Champlain, a total ascent of 140 ft. and a descent of 43 ft. The Oswego Canal descends 119 ft. from its junction to Lake Ontario. The Cayuga and Seneca Canal has a total ascent of 71 ft.

The channel of the waterway has a uniform bottom width of 75 ft. in earth sections of the land line, 94 ft. in rock, and 200 ft. or more in the beds of rivers and lakes, and has been excavated to a depth of 12 ft.

The locks are of the miter-gate type and have a usable width of 44½ ft. and a length of 300 ft. with a minimum depth of 12 ft. over the miter-sills. There are thirty-four locks on the Erie, eleven on the Champlain, seven on the Oswego, and four on the Cayuga and Seneca Canals.

The water supply for the western half of the Erie Canal is obtained from the Niagara River and for the eastern half an adequate supply is assured through the operation of two storage reservoirs which feed into the summit level. These reservoirs have a total capacity of more than 6 000 000 000 cu. ft. and are located at Delta on the head-waters of the Mohawk River and at Hinckley on West Canada Creek, one of the branches of the Mohawk. The Champlain Canal is dependent on the waters of the Hudson River for its supply, and the Oswego and Cayuga and Seneca Canals on the rivers that were canalized to form these canals and are fed by the lakes in the central part of New York State. Dams of both the fixed and movable type have been built as a means of controlling the canalized rivers.

The canal is crossed by about 306 railroad and highway bridges, the greater number of which are of the fixed type having a minimum clearance of $15\frac{1}{2}$ ft.

Terminals have been constructed at every city and nearly every village along the line of the canal. The facilities at the several sites vary, but in general consist of docks, wharves, harbors, freight-sheds, mechanical devices, and, in some cases, railroad connections for the interchange of freight between railroad and water carriers. At Gowanus Bay, Brooklyn, and at Oswego, on Lake Ontario, modern grain elevators have been constructed with capacities of 2 000 000 and 1 000 000 bushels, respectively.

To December 1, 1925, the State of New York has spent in the construction of the Barge Canal System and its terminals the sum of $174 258 558, which does not include interest on bonded indebtedness or maintenance cost, which now averages about $7 000 000 per year. So much for the physical structure itself.

Economic Features

The importance of the territory adjoining the Barge Canal is not generally appreciated. More than 70% of the people of New York State live within 2 miles of the waterway, which means that 7% of the population of the United States is within a 30-min. walk of the New York State Waterway System. It is easy to understand what it means not only to the State of New York but to the country at large that the products of these 8 000 000 people and the supplies they need shall have available a cheap means of transportation.

The Canal System as it exists at present has an annual carrying capacity in excess of 20 000 000 tons. During the navigation season of 1925, it carried 2 333 000 tons, which was 15% in excess of the tonnage carried in the highest previous year and a gain of more than 100% over 1918, the first year the Barge Canal was placed in operation throughout its entire length.

This question naturally suggests itself, why is the New York State Barge Canal System carrying only 10% of the tonnage it is capable of moving? Almost every one seems to feel qualified to answer this question but rarely do two people arrive at the same conclusion. In the attempt to diagnose this case one can proceed without fear of contradiction on the accepted basis that water transportation is inherently cheaper than any other form and that the territory served by this waterway system has a potential tonnage suitable for shipment by canal far in excess of the canal's present capacity.

Development of Traffic

Irrespective of the good physical condition of the canal or of its proved serviceability, carriers must be present to transport the tonnage if the full benefits of water-borne transportation are to be realized. There is a lack of boats on the New York State Canal System; of this there can be no question. The situation is not unlike that of a railway system which, after having provided its right of way, constructed the roadbed, laid its rails, built its terminals and freight houses, only awaited the arrival of the rolling stock to make it a going concern. Back in the Seventies when the 7-ft. canals were handling 6 000 000 tons of freight annually, there were on the canals between 5 000 and 6 000 boats of 240 tons capacity, drawn by horses and mules. To-day it is

doubtful if there are 800 boats capable of service and in condition to meet the requirements of the underwriters. Why this great decrease in the number of boats?

The agitation for a larger canal started in the Nineties, but it was not until 1903 that the dimensions of this larger canal were decided. During these years the boat owner was aware that sooner or later a canal would be constructed of a size on which the 240-ton boat then in operation would not be the most efficient unit, and naturally he spent nothing on new equipment and the minimum on upkeep and repairs. The canal was fifteen years in the building and during that period the number of boats maintained in the canal service became less and less each year. The boat owners and operators were still undecided as to the proper type of boat that could be used most efficiently on the new waterway—and even to-day there is a wide difference of opinion—which indecision resulted in a still further loss of boats in the service and the abandonment of those which had outlived their period of usefulness or required any extensive repairs. No new boats were constructed to take their place. The World War came on with a great increase in the prices of labor and materials. Priorities could not be obtained for materials for building canal boats. The Government took over the operation of the canals and for a time no assurance could be obtained that privately constructed boats intended for service on the New York Barge Canal would not be commandeered by the Government and placed on some other waterway. Conditions were such immediately following the close of the war that boat-building operations were not stimulated. Then came a period of great agitation for a still larger canal—a ship canal—following the St. Lawrence route or the Oswego-Mohawk-Hudson route through New York State. Those operating on the present canal or those contemplating such an operation found themselves somewhat in the same position as their predecessors prior to 1903. Should a larger and different type of canal be constructed the boats best adapted for use on the present canal probably would not be the most efficient for use on the new waterway and, therefore, operators rather felt that they should curtail building operations and await developments. Finally, the creation of a State Commission to make a survey and study of the Canal System with a view of recommending what should be done to stimulate traffic, gave rise to the thought in some quarters that New York State might abandon its canals. The net result of all this has been that capital seems reluctant to finance boat-building operations. In the writer's judgment this lack of carriers is the main reason why the Barge Canal is not transporting the tonnage for which it was designed and to-day is capable of carrying.

Types of Carriers Used

An outstanding example of the use that can be made of the Barge Canal is furnished by the Standard Oil Company. This Company is now operating on the Canal System nine steel tank barges the largest of which are 240 by 40 ft. and, on a 9-ft. loaded draft, carry 14 000 barrels of gasoline. Tanks have been constructed at many of the cities and villages along the line of the canal, and the oil or gasoline is pumped directly from the barges to these tanks and distributed by motor truck to the adjacent territory. This Company is con-

stantly extending the scope of its operations; in 1925, tank barges were operated as far west as Buffalo, south to Ithaca on Cayuga Lake, north to Ogdensburg on the St. Lawrence River and to Rouses Point at the northern end of Lake Champlain. The Standard Oil Company has built its barges to fit the canal and the fact that each year additional barges are put in service and land-station facilities are extended furnishes the answer as to whether the use of the Barge Canal is looked on with favor by this Company.

Other types of craft operating on the Barge Canal include the two steel motor ships of the Minnesota Atlantic Transit Company. These have a length of 258 ft., a width of 42 ft., and operate on a 10-ft. loaded draft. They are classified for Great Lakes and coastwise service and during 1925 have been operating between New York and Detroit, Mich. Loaded to 10 ft., they have a carrying capacity of 1 200 tons and make the trip between New York and Oswego, on Lake Ontario, a distance of 338 miles, in 72 hours.

The Interwaterways Line operates five steel motor ships, 256 ft. long and 36 ft. beam, which, on a 9-ft. loaded draft, carry about 1 500 tons. These ships do not operate on the Great Lakes, nor in coastwise service. The running time of these boats between New York and Buffalo, a distance of 507 miles, is between 5 and 6½ days. A round trip from New York to Buffalo, including the discharge and reloading of cargo at Buffalo, has been made in 11 days and 15 minutes. The cost of these boats is about $175 000 per unit.

The Trans-Marine Line operates its boats in fleets, the equipment consisting of thirty steel barges, five wooden barges, and five Diesel engine tugs used for towing the barges. The barges have a length of 100 ft. and a beam of 20½ ft. Five barges and one tug comprise a fleet which, on a 9½-ft. loaded draft, has a carrying capacity of about 2 000 tons. The cost of such a fleet is about $150 000.

The Munson Line has recently absorbed the New York Canal and Great Lakes Corporation, operating seventy steel barges built by the Government during the war. These barges are 150 ft. long and 20 ft. beam. Nineteen of these boats are power boats with only part cargo-carrying capacity and they tow the other barges. Four boats fill a lock and on a loaded draft of 10 ft. have a carrying capacity of about 1 950 tons.

The Brooklyn and Buffalo Navigation Company operates eighteen wooden barges and three cargo-carrying steamers. The barges are 100 ft. long and 22 ft. beam, and a fleet consisting of five barges and the steamer utilizing the full capacity of the lock, on a draft of 10 ft. 4 in., has a carrying capacity of 3 600 tons. The cost of such a fleet is about $100 000.

The statements made relative to the different types of boats are for the purpose of identifying the principal types of craft now operating on the Canal System and do not cover all the operating companies or the individual or independent operators who are using the canal.

Physical Conditions

Much misinformation has been spread broadcast relative to the physical condition of the Barge Canal, particularly as to the channel depth. The project depth was 12 ft., and the canal was excavated to that depth. Engineers

can readily appreciate the impossibility of maintaining a 12-ft. channel for full project width at all times on a canal which at the outset was dredged to only 12 ft., and which for the greater part of its length is located in the beds of natural watercourses. The original designers had in mind a canal to float boats on a 10-ft. loaded draft and fixed the depth to which the channel should be excavated at 12 ft. Unfortunately, the impression prevails that if boats in a 12-ft. channel cannot be loaded to a draft of approximately 12 ft., there must be something wrong with the canal. Observers state that there is insufficient depth, but cannot give its exact location. Testimony of operators has been given before the Barge Canal Survey Commission showing that in 1925 boats drawing 10½ ft. of water have navigated the canal from Buffalo to Troy without difficulty. Even in the face of such proof as to the absence of obstacles in the channel one hears on all sides that if the canal were dredged to its statutory depth boats in greater number would navigate it. During the latter part of the navigation season of 1925 the public press carried statements to the effect that the depth of the canal was only 8 ft. in certain sections. This was not so. At the time the writer knew of no place in the channel between Buffalo and Troy at which a boat drawing 10 ft. could not safely navigate, and such was the contemplated loaded draft for vessels when the decision was made to construct a 12-ft. canal. During 1924 and 1925 the policy has been adopted in removing material under the head of "Maintenance" to excavate 2 ft. below the established grade line, thus affording a channel depth of 14 ft. There is nothing more detrimental to the progress of the canal than erroneous statements constantly being made as to the depth of the channel.

Rail Connections

The establishment of rail connections and pro-rating agreements with rail lines at the canal terminals are of the highest importance. The necessity of this for successful operation of the canal is obvious. All the railroads are not yet willing to look on canals as a medium of transportation, having a place and performing a useful function in present-day transportation system, and are not disposed to consider the canals as supplementing the other means of transportation. They are thus depriving the people of the benefits that would accrue from a co-ordination of rail, water, and truck movements. An undesirable condition is created by the New York Central Railroad Company which secured a Court order, preventing the enforcement of the order of the Interstate Commerce Commission for a connection and interchange between canal and rail at the Erie Basin Terminal, in Buffalo. This matter deserves prompt treatment.

Packet Service Possibilities

The consensus of opinion seems to be that the Barge Canal is adapted for packet service, that sufficient tonnage would be offered, and that such service properly conducted would be profitable. Packet lines, however, have not been organized and financial interests apparently are not anxious to finance such enterprises. That the State itself should try the experiment of a packet line has been advocated. Unquestionably packet service would offer a means

whereby thousands of people, residents of New York State, who contributed to the building of the canal system, might participate directly in the savings from water transportation. The State might not be going too far afield in trying the experiment, and by establishing a dependable packet service over a certain section of the canal and limiting the service to a definite period of time, might demonstrate whether the canal is adapted for such service.

Means for Increasing Usefulness

Placing the canals under a permanent commission has been strongly advocated and the sound argument advanced that a continuing policy for canal management under the direction of qualified engineers must be formulated if the canal is to succeed. The directing head should not change with every new administration. The State must "sell" the canal to the shipping public and by personal solicitation obtain business through personal contact the same as the railroad corporations.

There has never been a time when there was a more crying need for clear thinking in relation to waterways than at present. The writer is unalterably opposed to a penurious policy in the expenditure of money, if any additional expenditure for the New York State Canal System is necessary, but he is just as much opposed to the idea that regardless of what may or may not be accomplished any expenditure for waterways is by any and all means money well spent. Those who believe that waterways transportation is of a distinct value to the country should insist that waterways be not further burdened with questionable expenditures. The most careful investigation should prove beyond a shadow of doubt that the promised benefits are commensurate with the amount to be expended.

Regrettable as it may be, the Barge Canal is accepted as a failure by too many people and as something in which the people of New York State have invested millions of dollars from which they are not receiving any ample or proper return. Not a sufficient number of people participated directly in the savings in freight from the handling of 2 333 000 tons last year, and although it is estimated that the existence of the canal system resulted in a depression of rail rates in New York State in 1925 which saved railroad shippers $50 000 000, this indirect saving is not understood and the wonderful asset which the people of the State have in the canal system is not appreciated.

Although the writer realizes that on every occasion the people of New York State have generously voted money for canal improvements, it is nevertheless a fact that many are disappointed in the tonnage transported on the Barge Canal. The unsupportable presumption that canals in New York State are a thing of the past is becoming too pronounced. Before it is determined to ask the people to contribute toward the building of a different type of canal, however, every effort should be exhausted to make the present canal serve its intended purpose. The Barge Canal is not admitted by the people of the Middle West as an outlet to the sea, and unless it can be shown in the not too distant future that a canal of this barge type can be made an adequate outlet to the Atlantic there will be constructed, over some route, a ship canal from the Great Lakes to tide-water and the people of New York State will

be called on to pay substantially 30% of whatever is spent on such construction by the Federal Government. Therefore, the people of New York State deserve to have every agency used to make their existing canals a success.

Future Enlargement

In connection with studies now being made relative to a proposed ship canal following either the St. Lawrence River or the New York State route, the writer feels that consideration should be given to the possible benefits to be derived from deepening the present canal from 12 ft. to 15 ft. and increasing the width of channel to a minimum of 110 ft. This would allow boats which make full use of all available space in the locks to load to a depth of 11 ft. 9 in. and not be subject to interference or delays in navigating the canal. Self-propelled barges of more than 2 000 tons capacity could then efficiently operate on the canal and their carrying capacity would be increased from 20 to 30% with practically no increase in operating expense. Such a result would prove a greater inducement to capital to place boats on the canal, which must be offered or sooner or later the canal will be abandoned, for a canal to be successful cannot justify itself without boats.

The people of New York State, in view of the millions of dollars willingly voted for waterways, should have definite knowledge and absolute proof that some other type of canal will completely answer the demand before they are again asked to give the vast sum that will be New York's share of the enormous cost of a ship canal. Who can say that the deepening and widening of the present canal may not go a long way in affording the outlet to the sea which the people of the Middle West are now demanding.

RELATION OF DEPTH TO CURVATURE OF CHANNELS

Discussion*

By Edward N. Chisolm, Jr., M. Am. Soc. C. E.

Edward N. Chisolm, Jr.,† M. Am. Soc. C. E. (by letter).‡—A study has been made of the relation of depth to curvature of channels to ascertain whether the author's formulas and conclusions are applicable to the Mississippi River from Cairo, Ill., to Fort Jackson, La. The physical characteristics differ materially along this stretch, but in general terms the river may be divided into two reaches; Cairo to Red River, La., 772 miles in length, and Red River to Fort Jackson, 269 miles in length. Above Red River the stream is non-tidal, but below this point the slope and current are affected by the tide in the Gulf of Mexico.

It is common knowledge with those familiar with this stretch of river that a greater depth is found in the bend of the channels than in the straight reaches and that the water surface on the concave side of the channel is higher than that on the convex side. It is also well known that the water in the bends descends at the concave side of the channel, crosses over at the bottom, and comes up on the convex side, causing excessive erosion of the alluvial bank. Observations confirm the accuracy of the author's formulas where differences of head of about 1 ft. have been noted on opposite banks of bends with mean mid-section longitudinal slopes of about 0.4 ft. per mile. The results seem to indicate that the increased depth in bends is caused by the helicoidal movement of the water induced by the centrifugal force of the current on the concave side of the channel.

The modification of the Mitchell formula so as to make it of general application to all sizes of streams, to all degrees of curvature, and to every character of channel is worthy of note, and further consideration will be given the derived Formulas (4)§ and (5)§ to test the accuracy with which the various sections along the Mississippi may be computed, and the constant to be applied, if necessary.

Notation 6‖ states, in part, as follows:

"Whenever the radius of curvature is less than 40 times the square root of the area of the channel, no further deepening of the channel results from the increased curvature."

* Discussion on the paper by H. C. Ripley, M. Am. Soc. C. E., continued from March, 1926, *Proceedings*.

† Capt., Corps of Engrs., U. S. A., Fort William McKinley, Rizal, Philippine Islands.

‡ Received by the Secretary, May 6, 1926.

§ *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1909.

‖ *Loc. cit.*, p. 1910.

This seems to conflict with a recent comparison of cross-section elements made along this stretch of river. Practically all the prominent bends have a radius of less than 40 $\sqrt{\text{area}}$, or 18 000 to 20 000 ft., and the tabulated results show that some sections decreased and others increased, depending somewhat on the time the surveys were made, the stage of water, and whether the bend was sounded on a rising or on a falling stage.

The author's Corollary (2)* has been given consideration and practical applications were made to test its accuracy at various points along the river with the following results.

At College Point, La., 907 miles below Cairo, Section 996 of the cross-section elements was selected. Soundings taken in 1898 showed an area of 270 480 sq. ft. and again in 1921, an area of 229 900 sq. ft., which would indicate that a radius of about 20 000 ft. was necessary at this point to produce a non-destructive bend. The actual radius of the concave side of this bend is 4 300 ft. (scaled), or about one-fifth the radius computed by the formula for a bend classed as destructive. There is a slight erosion, but the bend has changed very little in a number of years and can hardly be classed as destructive.

At 81-Mile Point, 883 miles below Cairo, Section 849 of the cross-section elements was selected. Areas taken in 1897 and 1921 were 229 720 sq. ft. and 266 300 sq. ft., respectively. These areas would indicate that a radius of about 20 000 ft. was required, whereas the radius scaled from the chart is 3 300 ft., or about one-sixth the computed radius. The bend is not classed as destructive.

At Waverly Point, near Natchez, Miss., 705 miles below Cairo, Section 2415 had an area of 203 972 sq. ft. in 1895 and 254 730 sq. ft. in 1913, which would give a computed radius of 18 000 ft. to 20 000 ft., whereas the actual radius is 7 000 ft., or about one-third the computed radius, and the bend is not actually destructive.

At Marengo Bend, above Natchez, 696 miles below Cairo, Section 2388 had an area of 160 139 sq. ft. in 1895 and 193 620 sq. ft. in 1913, which would indicate a radius of 16 000 to 17 000 ft. necessary for a non-destructive bend. In this case, the concave side of the bend scaled 10 000 ft., or about five-eighths the computed radius, and the bend is very destructive.

At Giles Bend, 691 miles below Cairo, Section 2371 shows an area of 221 717 sq. ft. in 1895 and 195 893 sq. ft. in 1913, which would require a radius of about 18 000 ft. to form a stable bend. In this instance, the actual radius is about 13 000 ft. (scaled) ; this is considerably less than 40 $\sqrt{\text{area}}$ in 1895 and somewhat smaller in 1913 than in 1895-96, but the mean depth on this section increased from 40.0 to 45.7 ft. at bankfull stage and from 33.8 to 50.6 ft. at low water. The maximum depth also increased at low water from 62.0 to 105.2 ft. This does not agree with the statement that "no further deepening of the channel results from increased curvature."

At Kempe Bend, 673 miles below Cairo, Section 2304 had an area of 317 142 sq. ft. in 1895 and 269 796 sq. ft. in 1913, which would give a com-

* *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1910.

puted radius of about 21 000 ft. The actual radius is 9 000 ft., or about one-half the computed radius, and the bend is actually destructive.

At Hard Times Bend, 643 miles below Cairo, Section 2174, the areas were 188 891 sq. ft. in 1895 and 206 856 sq. ft. in 1913, which would give a computed radius of about 18 000 ft. The actual radius is 7 000 ft. (scaled) and the bend is destructive. At bankfull stage the mean depth was 30.8 ft. in 1895 and 32.8 ft. in 1913. At low water, the maximum depth was 65.0 ft. in 1895 and 54.2 ft. in 1913.

At Island 16, which is 117 miles below Cairo, Section 441 was selected with an area of 199 150 sq. ft. in 1902 and 155 358 sq. ft. in 1912, which would give a computed radius of about 16 000 ft. In this instance the actual radius is about 16 000 ft. and the bend is not destructive and agrees with the formula. At bankfull stage the mean depths were 34.5 ft. in 1902 and 26.6 ft. in 1912. The maximum depths at low water increased from 35.9 ft. in 1902 to 49.7 ft. in 1912.

Near Island No. 8, 47 miles below Cairo, Section 173 was selected with an area of 218 199 sq. ft. in 1902 and 216 933 sq. ft. in 1911, which would give a computed radius of about 18 000 ft. for a non-destructive bend. The actual radius is about 17 000 ft. (scaled) and somewhat less than 40 $\sqrt{\text{area}}$, and the bend is slightly destructive at some points and quite destructive at others. At bankfull stage, the mean depths were 28.6 ft. in 1902 and 31.1 ft. in 1911 and, at low water, the maximum depths were 46.0 ft. in 1902 and 47.9 ft. in 1911.

The localities used to test the application of the formula were selected at convenient points along the river and the results show that bends at College Point, 81-Mile Point, and Waverly Point, are not classed as destructive bends, whereas the bends at Marengo Bend, Giles Bend, and Hard Times Bend, are classed as destructive. In all the localities mentioned the radius of the bends was considerably less than 40 $\sqrt{\text{area}}$.

At Island 16 and at Island 8 the radius of the bends was about 40 $\sqrt{\text{area}}$. In one instance the bend was not destructive; in the other, parts of the bend were destructive and parts were not destructive.

Under the heading, "Conclusions",* the statement is made that "the maximum depth in the straight reaches of any stream is equal to the mean depth multiplied by a constant, and the maximum depth in bends is equal to the mean depth multiplied by the same constant plus the effect due to curvature". In order to verify the statement, the maximum and mean depths were taken from the computations of cross-section elements, Cairo to Red River and Red River to Fort Jackson.

A comparison of Stretches Nos. 14, 119, 143, 204, and 286, comprising 57 sections shows that the average maximum depth is 1.734 greater than the average mean depth on the straight reaches at low water, whereas a comparison of Stretches Nos. 8, 127, 128, 156, 197, 199, and 313, comprising 170 sections, shows that the average maximum depth is 1.841 greater than the average mean depth in the bends at low water. The results on this stretch—Cairo to Red

* *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1938.

River—appear to agree with the statement that the constant to be applied in the bends is greater than the constant to be applied in the straight reaches, but judging from the various constants found on the individual bends and straight reaches, the results are rather an arithmetical coincident instead of a law. The constant on the straight reach varied from 1.387 to 2.196 and on the bends from 1.519 to 2.019, which indicates a range of 25% or more in the constant to be applied in either case and a rather broad range to compute accurate results.

A comparison of Stretches Nos. 23, 37, 50, 95, and 111, comprising 110 sections, shows that the average maximum depth is 1.490 greater than the average mean depth on the straight reaches at low water, whereas a comparison of Stretches Nos. 9, 38, 51, 96, and 141, comprising 98 sections, shows that the average maximum depth is 1.744 greater than the average mean depth on the bends at low water.

The results on this stretch—Red River to Fort Jackson—seem to agree more uniformly with the formula and with each other. The constant in the bends is greater and the individual results are greater in general terms. The constant on the straight stretches varied from 1.362 to 1.627 and on the bends from 1.595 to 1.954, which shows more uniformity than on the non-tidal river above Red River.

The practical application of the results shows that there is a constant between the average maximum depth and the average mean depth, but the constant varies 25% on the river above Red River, which would not produce results of a high degree of accuracy. On the stretch below Red River, the constant is more uniform, and the results would be more accurate in its application.

EVAPORATION ON UNITED STATES RECLAMATION PROJECTS

Discussion*

By E. F. Chandler, Assoc. M. Am. Soc. C. E.

E. F. Chandler,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author deserves thanks for having summarized in such convenient and accessible form the results of so extensive a series of evaporation records from different localities. This illustrates that at any one station the evaporation has a definitely determinable average value, and that its total is nearly uniform at any one point, varying much less from year to year than the annual precipitation. This indeed might reasonably be expected, for, in most parts of the United States, rain actually falls during a very small portion of the year, only a few per cent. of the total. In most of the country a humidity of 100% is unusual, except when rain is falling, so that there is more or less rapid loss by evaporation nearly all the remaining time. Even if in very wet or very dry years the total hours of rainfall are doubled or halved, the resulting change in hours of the evaporation period will be an insignificant percentage.

Of course the evaporation at any station varies from hour to hour, day to day, and month to month, with temperature, wind movement, and humidity, but these factors are not so enormously different at the same point in annual averages or totals from year to year, and the total evaporation is fairly uniform.

The observed amounts are affected by the form and arrangement of the tank and its exposure, so that care must be given to the conditions if comparable records are to be obtained from different stations. The paper draws attention to the diversity of results between different stations even in near-by adjoining regions, so that figures must not be heedlessly transferred from one region to another and considered applicable without adjustment or modification. These divergences are caused not only by differences in form and exposure of tank, but primarily by differences in climatologic factors, and especially by altitude which necessarily has a great effect on evaporation.

It may be of interest, therefore, to submit a summary (Table 44) of another record of this same kind which was maintained through many years under the writer's supervision for the U. S. Geological Survey. This record was continued through every open season from April, 1905, to June, 1920, at the University of North Dakota, in Latitude, 47° 55′ N. and Longitude, 97° 04′ W.

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* Discussion on the paper by Ivan E. Houk, M. Am. Soc. C. E., continued from August, 1926, *Proceedings*.

† Dean, Coll. of Eng., Univ. of North Dakota; and Hydr. Engr., U. S. Geological Survey, University, N. Dak.

‡ Received by the Secretary, August 4, 1926.

The gauge was a floating metal tank, 3 ft. square and 18 in. deep, held in a raft so as to be protected from splashing. It was placed in a narrow pool, several acres in area, and 4 to 8 ft. deep, formed by a low dam on a small stream running through the level prairie of the University Campus. The water level is 820 ft. above sea level, and is about 10 ft. below the prairie level at a distance of 5, 10, or 20 rods from the stream. The water is at approximately the same level, and same temperature, within the tank and outside it. The exposure to wind is about the same as on any ordinary small pond or pool, although less than on a large lake; but, on the other hand, the lower layer of air has not become humidified as it would be by passing over a large lake. This record, therefore, is presumably a reasonably accurate basis for predicting reservoir or lake evaporation in this region.

TABLE 44.—SUMMARY OF RECORDS OF EVAPORATION GAUGE AT UNIVERSITY, NORTH DAKOTA, 1905 TO 1920.

Month.	Mean temperature, in degrees Fahrenheit.	Mean monthly evaporation, in inches.	Maximum evaporation recorded, in inches.	Minimum evaporation recorded, in inches.
January	3.6	....	....	....
February	6.5	....	....	....
March	22.4	....	....	....
April	42.0	2.69	....	....
May	53.4	4.35	5.77	3.48
June	62.9	4.94	7.09	3.22
July	67.4	5.69	7.01	4.83
August	64.9	4.94	7.03	3.94
September	56.2	3.63	4.24	2.32
October	43.5	1.98	3.32	1.29
November	26.1	0.68	....	....
December	10.6	....	....	....

The tank is refilled daily to a standard gauge mark, the loss by evaporation being accurately measured. Standard rain gauges of the U. S. Weather Bureau are within a few rods and, at the close of any day of rain, the difference between the recorded rainfall and the surplus to be removed from the tank to lower the water to the gauge mark is considered as the evaporation. The recorded final results are, therefore, the gross total evaporation, and are slightly more than one and one-half times the total normal annual precipitation of 20 in.

No authentic continuous records of evaporation were secured during the frozen season. The record was begun usually after the ice cleared from the pool, during the first half of April, and continued until permanent ice closure sometime in November. April and November records for most of the years are therefore, in part mere estimates, but only for that part of the month having the smallest evaporation. There were mid-season interruptions for a few days or weeks in one or two years also, but for most of the months the figures given in Table 44 are the averages for fifteen years.

The temperature given is the average of the standard maximum and standard minimum readings of each day at the adjacent station of the U. S. Weather Bureau, and from these the mean temperature for the entire year of 12 months

is 38.3° Fahr. The total evaporation for the 9-month period, April 1 to November 30 (as shown by these records, covering the whole time except the first and latest weeks of cold years presumably having small evaporation, so that the estimates for the omitted time will be fairly close), is 28.90 in. Winter records at other evaporation gauge stations would justify an estimate of slightly less than 3 in. for the four months not recorded, making an annual total evaporation of almost, but not quite, 32 in.

AEROPLANE TOPOGRAPHIC SURVEYS

Discussion*

By Haywood R. Faison, M. Am. Soc. C. E.

Haywood R. Faison,† M. Am. Soc. C. E. (by letter).‡—The first reaction to the study of Mr. Bergen's paper is profound admiration for his courage in undertaking a written analysis of the aerial photographic process. Like the solution of most engineering problems, demonstration in the field, shop, and laboratory is much more spectacular and convincing. Unfortunately, hitherto such demonstration has been available to but few engineers. It is to be regretted that there is no more dramatic way in which to broadcast this revelation than by leading the reader, step by step, through a logical sequence of equations, from axiom to axiom. The significance of the diagrams is grasped at a glance, but there is risk of losing the attention of the casual reader before spanning that tiny gap between the commonplace and the remarkable.

Introduction to the actual workings of the Brock process duplicates the experience of the skeptical urchin, confronted by Houdini; one witnesses the astonishing result before comprehending the means of its accomplishment. Only when later opportunity offers itself to manipulate and to experiment with the instruments of precision, to assist in bringing the reproductions to true horizontal and vertical scales, are the fundamental principles actually grasped and assimilated. One then experiences that inevitable twinge of disillusionment at the total absence of the miraculous.

The Green River Gorge through the Blue Ridge in Western North Carolina may not be ideal terrain for aero-mapping, but it is certainly unsuited to mapping by any other known method. Maps of parts of the region, by private surveyors, and by the U. S. Geological Survey, made throughout the last three decades, clearly reflect the disastrous limitations affecting even the best of topographic personnel and methods. The elements of time and cost prohibit that consistent adherence to a uniform standard of accuracy possible in aerial work. Local observations are likely to be unduly refined in certain zones, at the expense of adjacent inaccessible areas. Contour control points must usually be connected from memory, or by the aid of sketches, at best, rather than from the continuous stereoscopic relief under the eye of the aerial map draftsman throughout the operation.

While the aeroplane survey is free from these drawbacks, there are peculiar limitations to which the process is subject. Through a wide range of flying

* Discussion on the paper by George T. Bergen, M. Am. Soc. C. E., continued from September, 1926, *Proceedings*.

† Dist. Engr., Mees & Mees, Hendersonville, N. C

‡ Received by the Secretary, August 19, 1926.

altitude, the property of the reduction mechanism, which will be called precision-ratio, for lack of a truer designation, is equal to, or better than, that of an approved triangulation theodolite. It is evident that the errors to be corrected by this mechanism—those due to tilt, drift, and variations in the speed and altitude of the plane—may be assumed to be constant and independent of the "height-of-lens", although, in actual practice, flying near the ground tends to increase these errors. It would seem, therefore, that below a certain definite "height-of-lens" above the average ground elevation, the desired precision-ratio would be rapidly exceeded, tending to introduce serious relative inaccuracies into attempts at large-scale mapping for detail work. However, since the object of the Green River survey was to cover many square miles of difficult mountainous country, on a scale of 600 ft. to 1 in., showing possible sites for dams and other hydro-electric structures, for pond areas, and feasible routes for construction roads and transmission lines, the flying altitude allowed a scale-ratio well within the limits of the required accuracy, and permitted reasonable variation of contour interval as desired.

Among the many parts of the mapped region selected for detailed instrumental study, some were densely timbered. Nevertheless, the contours checked so closely as to bear out the author's contention that the ground surface was clearly visible "throughout the stereoscopic view". It is not clear whether the claim is made that magnification of the vertical dimension, under the stereoscope, increases the visibility through the interstices between the tree-tops. This would seem to involve a compensating distortion in the horizontal plane which would offset any such advantage in visibility. In any case, it must be admitted that large areas of heavy timber do exist, through which it would be impossible to see the ground surface, even stereoscopically, for any considerable distance.

Ground work was not stressed in the paper, and quite properly so, because an elaborate system is unnecessary. A high degree of accuracy in the ground control, or better still, a systematic check to avoid serious errors, is essential to the smooth working of the subsequent phases of the process. Detection of errors in the elevations of field points, and of inconsistencies in base-line measurements, is certain during the reduction of the plates to scale, but much laborious computation and comparison is required to locate the discrepancy beyond question of doubt, and to eliminate all elements of uncertainty.

If the fall of the leaves in autumn is allowed to intervene between the time of the exposures and the establishment of the ground control much confusion results from the ensuing obscurity or complete obliteration of the selected field points. Conversely, identification of the points, by their visibility from overhead, is made difficult for the ground party by the untimely budding of the leaves in spring. Peculiar light-reflecting properties of some materials, as caught on the photographic plates, must be taken into consideration by the ground party. Dark granite boulders, painted a brilliant white, were not to be distinguished from their brethren of natural complexion on the plates of the Green River survey. Definite points on the plates often proved to be indistinguishable on the ground, such as gravel and sand patches

of neutral hue. This always necessitated the choice of some other point in the immediate vicinity, sharply defined on both ground and guide print. Points on static shore lines, edges of solitary boulders, acute angles of road intersections, etc., afford ideal field points for elevation. The extremities, at least, of the base lines should be of approximately equal elevation.

As to the intricate shop and office work which embodies the principles of the new process, one must read between the lines of Mr. Bergen's orderly development of his analysis to sense the many disappointments, the groping in blind alleys, the discouragements which that determined group of pioneers finally surmounted. The subject touches activities in many branches of Civil Engineering, and the author's logic is sound and comprehensive. It is to be hoped that the paper receives the attention which it merits, because perfection and simplification of the process are being carried on continuously, and constructive comment from engineers at large may well be the basis for working out further improvements.

IRRIGATION DEVELOPMENT THROUGH IRRIGATION DISTRICTS

Discussion*

By O. V. P. Stout, Esq.

O. V. P. Stout,† Esq. (by letter).‡—This paper is a remarkably able and concise summary of a large subject. It leaves little in the way of discussion except an elaboration of the points summarized. It informs those who have not had contact with the subject and orients those who have had that contact.

Some emphasis may be added to the statements§ at the close of the paper, relating to a phase of peculiar personal interest to the average engineer, but scarcely unique or peculiar to the irrigation engineer or with irrigation districts. The writer has had some opportunity to observe that "the professional relationship between the engineer and his irrigation district client is still a more or less unstable one," and to speculate on the causes and the outcome. The cause is undoubtedly as stated, that "the human beings who constitute irrigation districts, like the rest of us, have not yet learned all the lessons of co-operation." The irrigation district, however, should be one of the best schools in which to learn co-operation, because it is imperative to the success of the district and to that of the individuals who compose it that the lesson shall be learned. Unfortunately, most or all of them are slow to learn. Fortunately, however, there are some indications that in time they do at least begin to learn.

In California, some of the older and larger irrigation districts seem to have reached that stage of progress in their learning at which a competent engineer-administrator of their affairs can be recognized, the affairs placed largely in his hands, and his tenure of position made fairly secure. Other districts, however, not all of them small nor young by any means, having for a period employed competent engineer-managers, have not only failed in appreciation of the work done by them, but also, on the basis of the little learning acquired in connection with the enterprise, have concluded that lay management of irrigation works is entirely adequate, and have dispensed

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* Discussion of the paper by E. Courtland Eaton and Frank Adams, Members, Am. Soc. C. E., continued from April, 1926, *Proceedings*.

† Div. of Agricultural Eng., U. S. Dept. of Agriculture, Berkeley, Calif.

‡ Received by the Secretary, July 28, 1926.

§ *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 433.

altogether with engineering advice and assistance. Others have not gone so far, but have placed the engineering, even on projects of some magnitude, in the hands of either very young men or of those whose engineering knowledge and training does not extend beyond the routine of the particular project in question.

In the absence of any immediate issuance of securities or the undertaking of important construction operations the State has no authority to compel engineering service to such districts.

FIELD PROCEDURE OF ADJUSTING THE GREAT CIRCLE LINE TO THE RHUMB LINE

Discussion*

By Messrs. C. V. Hodgson, D. E. Hughes, R. L. Faris, George L. Hosmer, and Joseph James Corti.

C. V. Hodgson,† M. Am. Soc. C. E. (by letter).‡—This method for reducing a great circle line to a rhumb line is evidently intended to apply primarily to lines of cadastral surveys for which the distances between control points are ordinarily not greater than six miles. Nevertheless, allusion is made to the application of the method to longer lines such as those found on National and State boundaries. In that connection, it may be of interest to point out some of the other considerations which must be taken into account by an engineer engaged in retracing such a line.

Practically all National and State boundaries, if defined by arcs of the parallel or of the meridian, were established by astronomical methods starting from a known point such as an astronomical station. A random line was first run by one of the usual methods for a number of miles and another astronomical position was then determined. The random line was then corrected to conform to the position of the new astronomical station. Although there are variations of this procedure possible, these general principles will apply to the great majority of cases.

The accuracy of the determination of the astronomical position at the new point will depend on the instruments and methods used. When the most accurate field instruments available are used with the proper methods, latitude can be determined with an accuracy of about 0.1″, which corresponds roughly to 10 ft. on the earth's surface. Longitude can be determined with an error of about 20 ft. With an engineer's transit, the error will perhaps be ten times as great. There may be, therefore, considerable error from this source to be considered when retracing or establishing a boundary line of great length.

Still another factor to be reckoned with has a much greater effect on the location of the boundary line than instrumental errors or detailed methods of establishing sections of a boundary line between astronomical stations, namely, the deflection of the vertical at the astronomical station. The existence of

* This discussion (of the paper by N. B. Sweitzer, Assoc. M. Am. Soc. C. E., published in May, 1926, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the opinions expressed may be brought before all members for further discussion.

† Asst. Chf., Div. of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.

‡ Received by the Secretary, July 1, 1926.

station error and its effect is known by all engineers, but they often lose sight of the magnitude of the error from this source.

Suppose that at two adjacent astronomical stations at which latitude is observed the deflection of the vertical is in opposite directions, throwing the latitude too far south in one case and too far north in the other. The relative deflection between the two stations affecting the latitude is, of course, the algebraic sum of the deflections in the meridian at the two stations. In country of moderate relief, the relative deflection between two stations, 50 to 100 miles apart, will not usually exceed 1″ to 2″, but in mountainous country or in regions where the rock structure varies greatly in density, the relative deflections may be several times that amount. Between the east and west coasts of the Philippine Islands, for instance, the astronomical distance is in error by more than 1 mile and a similar error exists between the north and south coasts of Porto Rico. In India at Dehra Dun there is a 12″ relative deflection between two points only 5 miles apart, which corresponds to an error of 1′ to every 100 ft.

The effect of these station errors is strikingly shown by an examination of State boundaries in the western part of the United States where the monuments have been connected to the first order triangulation of the Coast and Geodetic Survey. This triangulation is on the North American Datum (which means that the points have been computed on the spheroid of reference which seems best adapted to this part of the globe) and is connected to the initial datum point in Kansas. The latitude and longitude of this point were so selected as to reduce practically to a minimum the algebraic sum of the deflections of the vertical at astronomical stations symmetrically distributed over the United States. The maps and charts of the nation are based on the triangulation computed from this datum.

An arc of this first order triangulation crosses Kansas near the 98th Meridian. A boundary monument on the north boundary of the State was found to be 888 ft. too far north, while one at the south boundary was 500 ft. too far south, thus making the State about ¼ mile wider at this point than its intended constitutional limits. A boundary monument at the southeast corner of Montana was found to be 650 ft. too far south and the one at the northeast corner of Wyoming was 800 ft. too far south.

An error of that magnitude in the location of a boundary line is bad enough, but a still more disturbing effect from the viewpoint of the surveyor is that the amount and direction of the deflection of the vertical changes from point to point along the boundary line, so that a line established by astronomical methods is a zigzag line. A monument on the eastern boundary of Montana is 1 900 ft. too far east, while another one 150 miles south on the same boundary line and supposedly on the same meridian is 1 400 ft. still farther to the east. On the boundary between Nevada and Utah, two boundary monuments, 2° 10′ apart, are 1 040 and 3 300 ft., respectively, too far east. On the international boundary between the United States and Canada, along the northern boundary of Montana, two astronomical stations 98 miles apart showed a relative error in latitude of more than 1 400 ft., while

at another place on the international boundary, there is a relative error of 800 ft. in latitude in 9¼ miles.

The problem, then, of adjusting the random line between astronomical stations is complicated immensely by these station errors and the retracing of a line is correspondingly difficult. At once it is seen why there has been so much litigation over State boundaries. The only chance of making a new boundary line agree approximately with the old is to re-establish the astronomical stations first established, and to interpose no new ones.

The disadvantage of having boundary lines that are so difficult to re-establish cannot now be prevented, for no other method was available at the time they were first located. All political units are concerned, however, with the problem of maintaining those boundaries. This can be done in either of two ways: First, by a strong monumentation of the line and frequent inspection of the monuments as provided by some States; and, second, by a similar monumentation supplemented by triangulation along the boundary with a sufficient number of stations of the triangulation close to the boundary so that offsets can be measured to the boundary at critical points. Each station and reference point of the triangulation scheme thus becomes a monument from which the boundary can be re-established.

A large scale topographical map with parallels and meridians on the North American Datum, therefore, will show a State astronomical boundary meandering at a considerable distance from the parallel or meridian which it is supposed to follow. Since a boundary is legally fixed after its location is once accepted by proper authority, the only way to bring a boundary into proper relation geographically with the adjacent cadastral and topographic surveys mapped on the North American Datum would be for the States concerned to mutually agree to a re-survey, and that is usually impracticable. Good triangulation and topographical surveys along State and county boundaries, however, would be of immense benefit to the political units concerned, as well as to the land-owners adjacent to the boundaries.

D. E. Hughes,* M. Am. Soc. C. E. (by letter).†—The formulas of this interesting and instructive paper, are practically correct for use within the limits contemplated by the author. In Table 1‡ the angles to single seconds and the offsets to only the nearest link look inharmonious; but, as indicated by the author, ordinary transit readings will only approximate the computed values within 10 to 30″, which would not justify tenths of links; whereas, on the other hand, the odd seconds are useful in other calculations.

Probably no one would lay out a long parallel of latitude from a single great circle tangent, for the offsets determined by the simple formula and the distances of the mile-posts from the starting point would both become appreciably in error, except near the Equator. If he were only setting fence posts the engineer might choose to offset from such lengths of great circle arcs as would make the deflection angles whole minutes; but for setting mile-posts he would use miles of length, and, if on standard parallels, the six miles

* Civ. Engr., San Pedro, Calif.

† Received by the Secretary, July 17, 1926.

‡ *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions, p. 883.

that range lines are apart would be most convenient. Pointing out, as the author does, the likeness to railroad curves is helpful to many who may have assumed that to run a parallel of latitude is more difficult.

In olden times a student was shown that from any initial point a line of stakes, in Azimuth 90° or 270°, would, if continued, cross the Equator 90° away, and therefore would mark a great circle and not a parallel of latitude, which is the east and west line. Also, he would know that if the near-by stakes are cut off on a straight line tangent at the initial point their stumps will project above level ground varying amounts, each equal to the square of its distance from the initial point divided by the diameter of the earth (this reduces to 8 in. multiplied by the square of the distance in statute miles); and that in the absence of refraction this is the correction for curvature used in leveling.

Next, he would see that if on level ground other stakes are leaned against the sawed off stumps and pointed each at right angles to the axis of the earth, the leaning stakes will be in a plane parallel to the plane of the Equator, and that their feet on level ground are, therefore, on the parallel of latitude which embraces the initial point. As the angle between the plumb stump and the leaning stake is in each case equal to the latitude of the place, the offset on the level ground from the one to the other will equal the correction for curvature multiplied by the tangent of the latitude.

Later, after learning navigation, the student would see that an oblique rhumb line, if ever brought ashore, could be marked on the ground in the same way from a line of plumb stakes set to the corresponding azimuth; but that in this case the feet of the inclined stakes, although on the rhumb line, would not be at the same distances as the plumb stakes from the instrument, and would need to be moved along the rhumb line to positions square across from the line of plumb stakes, thus making new offsets which would be equal to the old ones multiplied by the sine of the bearing of the line. This then becomes equal to the correction for curvature times the tangent of the latitude, times the sine of the bearing, which, of course, is identical with the formula recorded by the author, except that he uses 1.01 links instead of 8 in.

The navigator, if given a transit instead of a mariner's compass on a wheelbarrow, will know that, to pursue the rhumb line with a succession of short transit lines, he must make deflections at their junctions which shall be proportional to their lengths, and produce an aggregate equal to the convergency of meridians. His calculation of convergency is about the same as the author's first formula on page 883,* wherein 6 miles is changed to 5.2 geographic miles, or minutes, and multiplied by both the sine of the course and the tangent of the latitude, which need be neither meridional latitude nor middle latitude when the line is so short.

The rhumb line, so simple to navigators, has been little more than a curiosity to surveyors, whose teaching and work have covered plane and geodetic surveying, including the marking of parallels of latitude as required

* *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions.

on some State boundaries and on standard parallels or correction lines in the public land survey. Of course, nearly all students were tricked to say that a point going 4 miles per hour on a loxodrome running N. 60° E. would never reach the Pole, in fact, this is still stated in the Standard Dictionary. Those who gave it a second thought, however, realized that every inch along this course, N. 60° E., would make a northing of ½ in., and that the point would therefore reach the Pole as quickly as would another going due north at one-half the speed. This is a pretty illustration of an infinite series decreasing so rapidly that its sum is finite and moderate; also of the fact that although all lines reaching the Pole must come from the south in the geographer's language, yet the navigator or surveyor at the Pole may arbitrarily assume any one of them as reference, and from it turn off any course desired.

It is startling, however, to old Californians to read that farm lines, railroad tangents, etc., are "in the great majority of cases" loxodromic curves, and that such a curve is the "legal line"; for they had tried hard to keep the lines horizontally straight.

Suppose that from an existing corner, say, in Latitude 43° N., a grant line is recorded as having been run 47° E., 6 miles, to a tree now gone. A rhumb line N. 47° E. would miss the stump by a rod; another rhumb line bearing N. 47° 01′ 47″ E. would hit it; but there is no record of such bearing. Furthermore, this new line would have a 4-ft. bow, and the ranchers and the Courts want the fence line straight. Of course, an engineer seeing N. 47° E. written on a line on a plat, can advance a technical interpretation that all parts of the line may be presumed to have that bearing, and therefore that each small part must so bear with respect to its own meridian, and thus make a rhumb line. However, in California at least, the ranch was defined by plane surveying, with bearings referred to one meridian, through the point of beginning unless there was need to adopt a faulty one to fit an adjoining ranch.

When the magnetic compass was used to initiate a course the effort afterward was, by aid of back-sight or fore-sight, or both, to keep the line straight; and on long lines the varying needle bearings were read so that the last instead of the first could be used if needed at the next set-up, and thus eliminate diurnal variation of declination, local attractions, and convergency of magnetic meridians. When the solar attachment was used, and if along the line new meridians were determined by it, deflections were made to keep the lines straight and the corner angles correct, except when running standard parallels. It should not be lightly assumed that the old surveyor neglected to allow for convergency of meridians. The best mathematicians and most careful thinkers lived long ago. Even were it necessary now to assume that the old surveyor had failed to allow for convergency of meridians, yet it would not be warrantable to assume that he had traced a rhumb line instead of a succession of great circle arcs; and if it is not known at what stations he was guided anew by the sun alone, there is no telling whether his arcs were nearer a rhumb line than a single great circle.

For comparison of methods, assume a triangular tract in latitude 43° N. with the boundary described as beginning at Station 1, witnessed so and so; thence N. 47° E. 8 915 ft. to Station 2; thence S. 65° E. 14 387 ft. to Station 3; thence W. 19 559 ft. to point of beginning. In that latitude each 1 000 ft. of northing or southing changes the latitude 9.87″; in each 1 000 ft. of easting or westing, the longitude changes 13.53″; and the convergency of meridians is 9.20″. Here, the meridians at Stations 1 and 2 converge 1′ and at Stations 2 and 3, 2′. Hence, the geodetic surveyor would record azimuths as follows:

Stations 1 to 2, 227°; Stations 1 to 3, 270°; Stations 2 to 1, 47° 01′.
Stations 2 to 3, 295° 01′; Stations 3 to 2, 115° 03′; Stations 3 to 1, 90° 03′.

He, however, would agree with the plane surveyor in restoring Corners 2 and 3 and the intermediate line marks, for both surveyors would orient the same at Station 1, turn the same angles, and run the lines horizontally straight.

But how will the rhumb surveyor restore the corners? If he finds the old surveyor had a solar compass, will he assume he used it incorrectly? And if so, will he use the recorded bearings, which if applied to rhumb lines, will place Station 2 about 1.3 ft. to the northwest and Station 3 about 8.5 ft. north of the former positions? Or will he run his lines on calculated substitute bearings of N. 47° 00′ 30″ E., S. 64° 58′ E., and N. 89° 58′ 30″ W., in order to hit Corners 2 and 3? Even then his corner angles will not be correct, being 111° 58½′, 25° 00½′, and 43° 01′, instead of 112°, 25°, and 43°; his fences will not be straight; and he will have violated the general understanding and the tenor of Court decisions, that lines from point to point are to be straight, horizontally, unless specifically described as following a parallel or other curve.

This discussion is admittedly elementary, not for old men who will not read it, but for some young man who may be prone to apply hastily seemingly new things too widely. It is not a criticism of the author's paper which is clear in giving a simple and correct method of locating a parallel or other rhumb line of moderate length. The author may yet explain more definitely when to use rhumb lines instead of great circles in retracing lost boundaries; and he will be likely to agree that in case of doubt it is safer to use the latter.

R. L. Faris,* M. Am. Soc. C. E. (by letter).†—A consideration of the general formulas developed in Helmert's "Geodesy" shows that Mr. Sweitzer's formulas are approximations that are sufficiently accurate for practical use if the distance taken is not much more than that indicated in the paper. When the rhumb line does not coincide with the parallel it will of course not be a circle, and except when it coincides with a meridian or the parallel it will not be a plane section of the ellipsoid. However, for the distance treated in the paper the approximation to a circle is sufficient to answer all practical purposes and the field procedure suggested is justified. The attempt to express

* Asst. Director, U. S. Coast and Geodetic Survey, Washington, D. C.

† Received by the Secretary, July 27, 1926.

the results of general formulas in simpler form for practical use is commendable and no doubt saves engineers much time and trouble in practice.

The writer finds in his field notebook (dated 1892), a discussion of this same problem which appears to be shorter and probably somewhat more approximate, but which gives practically the same results. By way of comparison this old method may be of interest.

Inclination of the Meridian.—In Fig. 2 let AG and BG be arcs of a quadrant of meridians, 1° apart in longitude; then $AB = 1°$ of longitude on the Equator $= 69.16$ statute miles. Let DE be an arc of longitude on any parallel of latitude, and EH and DH the tangents meeting the earth's axis, produced at H, and corresponding to the parallel, DE. Then the line, $EF = DF = \cos\phi = \cos AD$, or BE; $DFE = 1°$; and the angle, $DHE =$ inclination of the meridian, which will be called X degrees.

Required to find an expression for X degrees. The triangles, FDE and DHE, have the same base, ED, and are isosceles; hence their vertical angles vary as their sides (vertical angles, DFE and $DHE = X°$). Hence,

$$1° \times EF = X° \times EH$$

but $EF = \cos\phi$, $EH = \cot\phi$, and the angle of $1° = DFE$, hence,

$$X° \cot\phi = 1° \cos\phi$$

and,

$$X° = \frac{1° \cos\phi}{\cot\phi} = 1° \sin\phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

that is, the inclination of the meridians for any difference of longitude varies as the sine of the latitude.

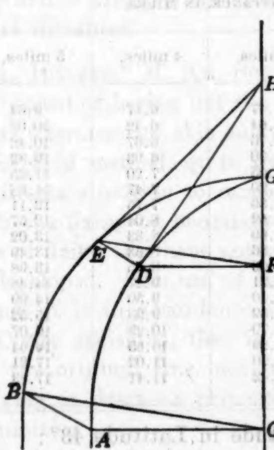

FIG. 2.

FIG. 3.

To get the inclination in seconds of arc, multiply by 3600 the number of seconds in 1 degree. The above is the deflection for 1° of longitude. To get the deflection for 1 mile, divide by the number of statute miles in 1° of longitude at the particular latitude under consideration.

Thus, in Latitude 43° (Example 1):

$$\begin{aligned} \log \sin \phi &= 9.833783 \\ \text{“ } 3600 &= \underline{3.556303} \\ & 3.390086 \end{aligned}$$

TABLE 2.—Inclination of Meridians.

Latitude, in degrees.	Inclination for:		Deflection of Tangents:	
	1 mile.	6 miles.	1 mile.	6 miles.
30	30.03″	3′ 00″	15.02″	1′ 30.0″
31	31.26″	3′ 07″	15.63″	1′ 33.5″
32	32.49″	3′ 15″	16.24″	1′ 37.5″
33	33.83″	3′ 23″	16.92″	1′ 41.5″
34	35.17″	3′ 31″	17.58″	1′ 45.5″
35	36.50″	3′ 39″	18.25″	1′ 49.5″
36	37.83″	3′ 46″	18.92″	1′ 53.0″
37	39.17″	3′ 55″	19.58″	1′ 57.5″
38	40.67″	4′ 04″	20.34″	2′ 02.0″
39	42.17″	4′ 13″	21.08″	2′ 06.5″
40	43.67″	4′ 22″	21.84″	2′ 11.0″
41	45.17″	4′ 31″	22.58″	2′ 15.5″
42	46.85″	4′ 41″	23.42″	2′ 20.5″
43	48.52″	4′ 51″	24.26″	2′ 25.5″
44	50.19″	5′ 01″	25.10″	2′ 30.5″
45	52.00″	5′ 12″	26.00″	2′ 36.0″
46	53.83″	5′ 23″	26.92″	2′ 41.5″
47	55.67″	5′ 34″	27.84″	2′ 47.0″
48	57.67″	5′ 46″	28.84″	2′ 53.0″
49	59.83″	5′ 59″	29.92″	2′ 59.5″

TABLE 3.—Offsets, in Feet.

Latitude, in degrees.	Distance, in Miles.					
	1 mile.	2 miles.	3 miles.	4 miles.	5 miles.	6 miles.
30	0.39	1.54	3.47	6.17	9.64	13.88
31	0.40	1.60	3.61	6.42	10.03	14.44
32	0.42	1.67	3.76	6.67	10.42	15.02
33	0.43	1.73	3.90	6.93	10.82	15.60
34	0.45	1.80	4.05	7.20	11.25	16.20
35	0.47	1.87	4.20	7.47	11.68	16.81
36	0.48	1.94	4.36	7.75	12.11	17.41
37	0.50	2.01	4.52	8.04	12.57	18.09
38	0.52	2.08	4.69	8.33	13.02	18.75
39	0.54	2.16	4.86	8.63	13.49	19.43
40	0.56	2.24	5.03	8.95	13.98	20.11
41	0.58	2.32	5.21	9.27	14.48	20.85
42	0.60	2.40	5.40	9.59	14.99	21.59
43	0.62	2.48	5.59	9.93	15.52	22.35
44	0.64	2.57	5.79	10.29	16.07	23.14
45	0.67	2.66	5.99	10.65	16.64	23.96
46	0.69	2.76	6.20	11.02	17.21	24.80
47	0.71	2.85	6.42	11.41	17.83	25.68

Divide by 50.66 miles = 1° longitude in Latitude 43°:

$$\begin{aligned} \log &= 1.704682 \\ & \underline{3.390086} \\ &= 1.685404 \text{ (in seconds)} \\ &= 48.''\,46. \end{aligned}$$

which is the inclination for 1 mile of longitude.

Offsets.—In Fig. 3 let $H\ D$ and $H\ E$ be tangent to two meridional arcs at D and E as in Fig. 2.

Let $T\ N$ be a tangent to the arc of parallel at the point, D; then is it an east and west line at D. Then the angle, $E\ D\ N$, is the "angular deflection" in running the distance, $D\ P$, beginning at D on a true east and west course; and $P\ E$, perpendicular to $T\ N$, is the "offset" from the arc of the parallel, at E.

The angle at H, or the angle that measures the inclination of the two meridians $= \dfrac{\cos \phi}{\cot \phi} \times 3600$ for 1° of arc (Equation (1)).

The angle at $H = 2$ Angle $E\ D\ N$; or the angle of meridian inclination is twice the angle of deflection of an east and west line from an arc of parallel at the point, D. Hence,

$$\text{Angle } E\ D\ N = \frac{1}{2} \text{ Angle } H$$

The "offsets", such as $E\ P$, are computed from the distance (length of course), such as $D\ P$, and the angle of deflection, such as $E\ D\ P$. Thus, from Example 1, the angle of inclination for 1 mile, Latitude 43° = 48.″ 46; hence,

$$\text{Angle of deflection} = \frac{48.''\ 46}{2} = 24.''\ 23 \quad \log \tan = 6.0698637$$

$$1 \text{ mile} = 5280 \text{ ft.}; \log = \underline{3.7226339}$$

$$\log \text{offset} = 9.7924976$$

$$\text{Offset} = 0.62 \text{ ft.}$$

Tables 2 and 3 give the inclination of meridians and offsets for various latitudes and distances.

George L. Hosmer,* M. Am. Soc. C. E. (by letter).†—This paper, describing a field method of laying out the loxodromic curve, is a substantial contribution to the literature of this subject, not only because it presents a simple and practical field method, quite within the reach of all engineers, but also because it directs attention to much needed improvements in the methods of dealing with the fixing of boundary lines of all sorts. In the United States the methods of doing this have not received much attention and have not been very fully developed. The use of the loxodrome as the legal line is perfectly sound because it is in accordance with the accepted principle and the most important Court decisions, that in a re-survey the object is to recover the position of the original line, not to correct mistakes which may have been made in laying it down—a principle which has not yet been discovered in some communities.

The author's plan of substituting a circular arc, or a series of circular arcs, for a portion of the loxodrome, is sufficiently accurate and gives practically the same line on the ground. Such curves can be dealt with in a manner familiar to all engineers. The short offsets from the geodetic line to the

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† Received by the Secretary, July 29, 1926.

loxodrome are given by his formulas to the nearest hundreth of a foot in the latitudes of the United States.

The geodetic formulas first mentioned in this paper are, of course, the Puissant formulas as adapted for use on the United States Coast Survey by Hilgard about 1846. The one that applies most directly to this computation is that giving the convergence of the meridians, or change in the direction of the loxodrome, namely,

$$-d\,Z = d\,\lambda \sin \tfrac{1}{2}\,(\phi + \phi')$$

The author has used this same formula* in a slightly modified form.

The formula, $5'.2 \tan \phi$, is based on a 6-mile distance. At the equator this would subtend an angle of $5'.2$. The difference in longitude on any other parallel of latitude would be $5'.2 \times \sec \phi$. This is the difference in longitude just as it would be computed by a navigator, the $5'.2$ being the departure in nautical miles. This amounts to computing the difference in longitude for a particular distance along the curve, and the process is equivalent to the solution of the second equation in the group referred to. The convergence of the meridians (equal to the change in direction of the loxodrome) is given by:

$$\begin{aligned} d\,Z &= 5'.2 \times \sec \phi \times \sin \phi \\ &= 5'.2 \times \tan \phi \end{aligned}$$

Along a curve having a small course the change in latitude must be allowed for, since it affects the value of $\tan \phi$. The difference in latitude is given by the distance expressed as arc and multiplied by the cosine of the course. This is equivalent to solving the first term of the first equation, which is the only large term of this series. In dealing with any line not lying nearly east and west it is necessary therefore to solve all three of the geodetic equations, in one form or another, although this may not be obvious from the simple equations stated by the author.

These three equations may be re-stated in a simple form, sufficiently accurate for the purpose in hand, as follows: Assuming the earth to be a sphere, a statute mile subtends an angle of $0'.868$. (The suppression of the terms due to the ellipticity of the meridian appears to cause an error of less than 0.5%, in the latitudes of the United States.) The three equations become (in the notation already used):

$$\begin{aligned} -d\,\phi &= 0'.868 \times D \times \cos \theta \\ d\,\lambda &= 0'.868 \times D \times \sin \theta \sec \phi_m \\ -d\,Z &= d\,\lambda \sin \phi_m \end{aligned}$$

in which, $d\,\phi$, $d\,\lambda$, and $d\,Z$ are the differences (in minutes) of latitude, longitude, and azimuth; D is the distance, in statute miles; θ is the course, or bearing; and ϕ_m is the mean, or middle, latitude.

It is natural that the use of the loxodrome as the true line should be adopted in regions surveyed by the United States System, since this curve forms the basis of the system. In the Eastern States as a rule the loxodrome has not been adopted as the true line. The original surveys of certain boun-

* *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions, p. 882.

daries were run out by transit (some of them as early as 1787), and are therefore properly treated as straight, or "geodetic", lines. Others, although run out by the use of the magnetic compass, have been treated subsequently as if they were straight between fixed points. If the loxodrome were to be used for such boundaries, or portions of them, the preceding formulas might have to be slightly modified, for convenience; but there is no reason for not retaining the fundamental idea, that is, to regard the loxodrome as the true curve, but to substitute the circular curve for a portion of it in the field operations covering comparatively short arcs.

JOSEPH JAMES CORTI,* ASSOC. M. AM. SOC. C. E. (by letter).†—The problem of locating a loxodrome, or rhumb line, by means of a series of chords, can be solved, south of the Equator, through the use of formulas similar to those given by the author, as follows:

$$d\phi = \phi' - \phi = B \;.\; K \;.\; \cos Z + C \;.\; K^2 \;.\; \sin^2 Z + D \;.\; h^2$$
$$d\lambda = \lambda' - \lambda = A' \;.\; K \;.\; \sin \;.\; Z \;.\; \sec \phi'$$
$$dZ = Z' - Z \pm 180° = d\lambda \;.\; \sin \phi_m$$

in which, ϕ is measured positively northward and λ, positively westward; and Z is measured counterclockwise from the north.

TABLE 4.—ITEMS FOR COMPUTING GEODETIC POSITIONS.

(Clarke spheroid, 1866, a = 6 378 206 $m.$; $c = 1 \div 295.$)

ϕ, in degrees.	log A' + 10.	log B + 10.	log C + 10.	log D + 10.	ϕ, in degrees.
0	8.50973	8.51268	$-\infty$	$-\infty$	0
2	8.50972	8.51267	9.950-10	1.23	2
4	8.50972	8.51265	0.252	1.54	4
6	8.50971	8.51262	0.428	1.71	6
8	8.50970	8.51259	0.555	1.83	8
10	8.50968	8.51254	0.654	1.93	10
12	8.50966	8.51249	0.734	2.00	12
14	8.50964	8.51242	0.803	2.06	14
16	8.50961	8.51234	0.864	2.12	16
18	8.50959	8.51226	0.918	2.16	18
20	8.50955	8.51216	0.967	2.20	20
22	8.50952	8.51206	1.013	2.23	22
24	8.50948	8.51195	1.055	2.26	24
26	8.50944	8.51183	1.094	2.29	26
28	8.50940	8.51170	1.131	2.31	28
30	8.50936	8.51157	1.167	2.33	30
32	8.50931	8.51144	1.201	2.35	32
34	8.50927	8.51130	1.234	2.36	34
36	8.50922	8.51115	1.266	2.37	36
38	8.50917	8.51100	1.298	2.38	38
40	8.50912	8.51085	1.328	2.39	40
42	8.50907	8.51070	1.359	2.39	42
44	8.50902	8.51054	1.389	2.39	44
46	8.50896	8.51039	1.419	2.39	46
48	8.50891	8.51024	1.449	2.39	48
50	8.50886	8.51008	1.480	1.39	50
52	8.50881	8.50993	1.510	1.38	52
54	8.50876	8.50978	1.542	1.37	54
56	8.50871	8.50964	1.574	1.36	56

* Civ. Engr., Mendoza, Argentine Republic.

† Received by the Secretary, August 18, 1926.

The factors, A', B, C, and D, are to be taken from Table 4, A' for Latitude ϕ', and B, C, and D for Latitude ϕ.

Table 4 has been prepared from "Formulae and Tables for the Computation of Geodetic Positions",* by reducing the number of factors and decreasing the number of decimal figures, both to the limits required by the field engineer. It can also be used north of the Equator, with slight changes in the formulas, as shown in the paper.

If from Point M, on Parallel 43° South, a loxodrome of 47° constant azimuth had been located, N and P being its intersections with meridians 5′ and 10′ west of M, the length and azimuth of the chords, $M\,N$ and $N\,P$, would be computed as shown in Table 5. Since the latitude of both points is known it makes no difference whether the computation is made first from M to N, and checked afterward, from N to M, or in the inverse order. However, if the loxodrome is to be located, the latitude of N being unknown, the computations must be made first from N to M, and checked by computing from M to N with the latitude obtained for N.

The computations from N to M are made as follows: First is computed $\log K \,.\, \sin Z = 3.83221\ n$, and $d\,Z = d\,\lambda \,.\, \sin \phi_m = 3'\ 24''.6$. It will be sufficient to take ϕ, the latitude of Point M, instead of the unknown latitude, ϕ_m.

The loxodrome is a curve which winds spirally on the surface of the earth, with curvature varying from point to point. For the purpose in view, however, the stretch between the ends of a chord may be taken as an arc of a parabola with the axis perpendicular to the chord at its middle point. This assumption fixes the position of the tangents at M and N, by taking $\frac{1}{2}\,d\,Z = 1'\ 42''.3$ as the angle that either of them makes with the chord, $M\,N$, and to write at the foot of Column (2) in Table 5, $Z' = 47°\ 1'\ 42''.3$ as the azimuth for $M\,N$, and $Z = Z' - 3'\ 24''.6 + 180° = 226°\ 58'\ 17''.7$ as the azimuth for $N\,M$.

Once Z is known, $\log \tan Z = 0.02991$, and by the set of operations,

$$(\log K \,.\, \sin Z = 3.83221\ n) - (\log \tan Z = 0.02991)$$
$$= (\log .\, K \,.\, \cos Z = 3.80230\ n)$$
$$(\log .\, K \,.\, \cos Z = 3.80230\ n) + (\log B = 8.51062)$$
$$= (2.31292\ n = \log 205\,552)$$

is obtained,

$$d\,\phi_1 = -\ 3'\ 25''.552$$

From $\log K \sin Z = 3.832$ and $\log C = 1.374$, m is computed, which in this problem will always have a minus sign; and this leads to $-\ 42°\ 56'\ 34''.557$ as the latitude of Point N.

If the computations were repeated giving to ϕ_m, B, and C new values in accordance with the latitude obtained for N, it would be found that the difference in the results would not justify the additional amount of work involved.

* U. S. Coast and Geodetic Survey, *Special Publication No. 8.*

TABLE 5.—EXAMPLE OF COMPUTATIONS.

Items. (1)		Computations. N to M. (2)		M to N. (3)		P to N. (4)		N to P. (5)	
λ	ϕ	5′	−42° 56′ 34.557″	0′	−43° 00′ 00.000″	10′	−42° 53′ 08.925″	5′	−42° 56′ 34.557″
λ'	ϕ'	0′	−43° 00′ 00.000″	5′	−42° 56′ 34.557″	5′	−42° 56′ 34.557″	10′	−42° 53′ 08.925″
$d\lambda$	$d\phi$	−300″	−3′ 25.443″	+300″	+3′ 25.443″	−300″	−3′ 25.632″	+300″	+3′ 25.632″
log $(K \sin Z)^2$	....	7.664	......	7.664		7.665		7 666	
log C	....	1.374	......	1.874		1.373		1.372	
log m	m	9.038	−0.109″	9.038	−0.109″	9.038	0.109	9.038	0.109
....	$d\phi_1$		−205.552″		+205.334″		−205.741″		+205 523″
log $d\lambda$	log $d\phi_1$	2.47712n	2.31292n	2.47712	2.31246	2.47712n	2 31332n	2.47712	2.31286
" A'	" B	−8.50904	−8.51062	−8.50904	−8.51062	−8.50904	−8.51662	−8 50904	−8.51062
" sec ϕ'	....	−0.13587	......	−0.13547		−0.13547		−0.13507	
log $K \sin Z$	log $K \cos Z$	3 83221n	3.80230n	3 83261	3.80164	3.83261n	3.80270n	3.83301	3.80224
" $\sin Z$	" $\cos Z$	−9.86393n	−9.83401n	−9.86433	−9.83356	−9.86393n	−9.83401n	−9.86433	−9.83355
log K	log K	3.96828	3.96829	3.96828	3 96828	3.96868	3.96869	3.96868	3.96869
K	K	9295 6m.	9295.8m.	9.295.6m.	9295.6m.	9304.2m.	9304.4m.	9304.2m.	9304.4m.
log tan Z	Z	0.02991	226° 58′ 17.7″	0 03077	47° 01′ 40.8″	0.02991	226° 58′ 17.9″	0.03077	47° 01′ 41.0″
" $d\lambda$	±180°	2.4771n	−180°	2.4771	+180°	2.4771n	−180°	2.4771	+180°
" $\sin \phi_m$	....	9.8338n	......	9.8336n		9.8330n		9.8331n	
" dZ	dZ	2.3109	+3′ 24.6″	2.3107n	−3′ 24 5″	2.3.01	+3′ 24.2″	2.3102n	−3′ 24.3″
....	Z'		47°01′ 42 3″		226° 58′ 16.3″		47° 01′ 42.1″		226° 58′ 16.7″

The computation of latitude must be carried to the third decimal place in seconds, in order that the disagreement in the values of azimuths may not exceed a few seconds.

The check from M to N is to be done entirely from the top downward, starting from the Latitude obtained when computing from N to M.

Lastly, four values of K will be obtained, two from the computations, M to N, and two from N to M; and the four values must check within a few decimeters.

If it were desirable to locate points of the parabola between the ends of the chord, the offsets would be determined by starting from the middle and longest one, given in meters by $\frac{1}{4}\ K \tan \frac{d\,Z}{2}$.

A comparison of the paper with the results reached by the writer, shows that the author has computed the values in Column (10) of Table 1* by means of the formula, $d\,\lambda = K\ .\ A'\ .\ \sec\phi$, with $K = 9\,656\ m = 1$ mile. From the values thus obtained Column (8) is computed by $d\,Z = d\,\lambda\ .\ \sin\phi$. With $d\,Z$ known, he computes Column (7) by $\frac{1}{4}\ K\ .\ \tan\frac{d\,Z}{2}$; and fills out Columns (2) to (6) with the offsets of a parabola.

The method of the paper, therefore, is shown to be correct; and the author is much to be commended for the preparation and publication of his Table 1, which so greatly simplifies the computations for problems of this type.

As to the location of a parallel, the combination of the tangent and the chord methods, makes it possible to simplify field operations, as will be developed subsequently.

After the direction of the chord and the tangent have been determined and after computing the offsets from each of these two lines to the parallel, along the meridians of the different points to be located, two surveying parties are sent to measure off the abscissas, one along the chord, the other along the tangent. When the two parties reach the foot of the two offsets for each point, the distance between these two must agree with the one computed, and the corresponding point on the parallel can be located with no other instrument than a tape.

For the location of 1° 40′ of the —36° parallel, part of the southern boundary of the Province of Mendoza, Argentine Republic, the computations were made with reference to two chords of 75 km. each, passing through the middle point of the arc toward the ends, and to the tangent touching the parallel at the middle point. The maximum distance between the foot of the two offsets for a common point was only 321.2 m.

* *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions, p. 883.

DISTRIBUTION OF REINFORCING STEEL IN CONCRETE BEAMS AND SLABS

Discussion*

By Messrs. Edward Godfrey and H. C. Sandbeck

Edward Godfrey,† M. Am. Soc. C. E. (by letter).‡—The portions of this paper that the writer would like to discuss concern the bending moments in flat slabs and the stirrup as a shear member.

Mr. Myers has shown that, taken as a standard or basis for computing bending moments, the value, $\frac{w\,l^2}{8}$, or the bending moment on a simple beam, has a peculiar place for different conditions of continuity or fixity of supports. On page 1109,§ he states:

"In all cases the sum of the center moment and average moment at the supports is equivalent to the moment for a simple beam of the same span."

While this is strictly and undeniably true, as can be readily demonstrated, there is not a standard in use for designing flat slabs that makes use of this principle or even approximates the reinforcement in the slab that this principle clearly demonstrates to be necessary.

Considering a flat slab as supported on lines of parallel columns and applying the principle just quoted, it is seen at once that the bending moment on a mid-section of the slab parallel with the lines of columns, added (numerically) to the bending moment on a section through the column centers, should be equal to one-eighth of the total load carried between the two rows of columns times the span between columns. No method of designing flat slabs provides reinforcement that even approximates this requirement.

If the lines of support of the flat slab are considered to be tangent lines touching the circles of the column heads, the case is simpler and the column heads and columns do not complicate the calculations. There is no code in use nor standard of design that provides reinforcement even approaching the requirements of the case as quoted.

There is no reason why flat slabs should be given this advantage over other slab designs. It is true that some tests on flat slab structures have appeared to show a strength that justifies the present standards. This is because those who have made these tests have never been willing to make a critical test on flat slab construction.

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* This discussion (of the paper by Boyd S. Myers, M. Am. Soc. C. E., published in August, 1926, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Structural Engr., Pittsburgh, Pa.

‡ Received by the Secretary, August 14, 1926.

§ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions.

Single bays or several selected bays, when tested in a flat slab floor, show great strength because the surrounding, idle floor aids the parts that are loaded. Spot loads that produce a dished shape in the floor slab do not test the floor critically. The critical load on a flat slab floor is a load that covers a row of bays clear across a building from wall to wall. A load of this sort deflects the floor in a cylindrical instead of a dished shape. The bending moment in the slab is quite definite under this loading, and it cannot be otherwise than that stated by Mr. Myers.

The flat slab problem is attacked from the standpoint of complex mathematics involving such factors as Poisson's ratio, and yet the simple criterion expressed in the paper is a definite and determining criterion which cannot be gainsaid. It reduces investigation of the flat slab, from the mathematical standpoint, to the simplest terms.

The other portion of the paper to which the writer would call attention is that part where the spacing of stirrups is derived. It is a *sine qua non* of any attempt to discuss the spacing of stirrups or to derive rules for that spacing, that one must first justify the stirrup and the basis of the method by which the rule of spacing is derived.

Mr. Myers mentions the paucity of rational methods of spacing stirrups in the literature of reinforced concrete. The real lack is a rational analysis of the stirrup itself. All treatment of stirrups in standard literature starts with the hypothesis that in some way, which it is not necessary to defend or explain, a stirrup will take an amount of shear corresponding to the vertical shear in the beam.

From the standpoint of analysis: It is many years since any authority has held that stirrups perform their function by resisting shearing stress in the steel itself, so that the idea of stirrups taking horizontal shear and transmitting this as increments into the horizontal rods has been abandoned. Shear in the steel rods is therefore eliminated from the analysis.

If the stirrup takes vertical shear as a compression stress, it would mean that the beam is shortened in its height from top to bottom. It is conceivable, if a layer of soft substance existed in a beam, in a diagonal direction, crossing a stirrup and sloping downward from the top of the beam toward the middle of the span, that compressive stress of considerable magnitude would be developed in that stirrup. A trifling amount of compressive stress could develop in stirrups due to the sag or deflection of fixed ended beams; but no authority mentions compression as the kind of stress for which stirrups are to be designed.

The stirrup must be considered, therefore, as a tension member. Regarding its action there are two types of analysis, both of which are merely fragmentary. One analysis shows a 45° section cutting the beam and the stirrups. The shear would be a tension on this diagonal section, and the analysis would divide this tension by the number of the stirrups cut to obtain the tension in one stirrup. This analysis ignores the fact that the section may cut through the very tip end of one or two of those stirrups, where the stirrup could have no anchorage or hold in the concrete. If the beam were actually severed in

this plane, these stirrups without anchorage would be useless, and yet they are given the same weight and value as stirrups that have some, if not a full, anchorage. In fact, few stirrups have anchorage enough to take the tension allotted to them in the design. To gain some idea as to the length needed for anchorage, suppose the stirrups are ⅜-in. square bars, and assume 40 diameters as the length needed for anchorage. The stirrup would require 30 in. of length for anchorage alone and only the middle of the length would be properly conditioned for its stress. It will be readily appreciated that stirrup and beam sizes do not conform to the dimensions that analysis thus shows to be necessary.

It will be said, however, that anchorage for stirrups is effected by looping and hooks rather than by embedment in the concrete. It has been demonstrated by tests that an ordinary loop does not constitute a proper and full anchorage for a steel rod. To assume that a loop of the stirrup effects this anchorage is to presuppose that the shear load is hung as a vertical load on the horizontal rod. The horizontal rod is not capable of carrying concentrated loads such as this.

It is these features of stirrup analyses that are ignored by authorities and that render the analyses of no value, since they do not follow the stresses out to adequate resistance.

The other analysis to which reference has been made is summed up in vague statements that stirrups act as the web members of a truss. Here, too, no effort is made to follow out these alleged web stresses to adequate resistance —to show how the stress is imparted to the stirrup and to what that stress is delivered after it passes through the stirrup. In truss analyses these matters are taken care of to the last pound in adequate details and sections; in reinforced concrete analyses they are totally ignored. What forces could there be in a beam stretching it out in a vertical direction to give tension in the stirrups?

From the standpoint of tests: No tests that the writer could ever find have shown anything but a trifling amount of compression in stirrups in a whole beam. Beams with diagonal shear cracks have exhibited high tension in the stirrups crossing these shear cracks; nothing else could be expected; but why did not the stirrups prevent the shear cracks, if they were performing their function? And why, in the whole beams, did the stirrups not show tension of considerable amounts rather than compression in trifling amounts, since it is tension for which they are designed? The confirmation of analysis is tests that at least approximate agreement with that analysis.

There is a space between stirrups where the full shear of the beam must be carried by the concrete. Between the last stirrup and the support this shear is the maximum, but there is no reinforcement in the beam of the standard design referred to by Mr. Myers to relieve the concrete of shear in this critical section. He specifically rejects reinforcement that would satisfy the requirement of steel reinforcement for all vertical sections under high shear, in these words, "Except in rare cases, the bent longitudinal bars should not be considered as shear reinforcement."*

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1119.

It is astonishing that Mr. Myers should reject the only shear reinforcement that is capable of any kind of analysis, which is at the same time a type of reinforcement that positively prevents a beam from failing. A beam that depends on stirrups or any kind of short shear reinforcement to take its shear may fail completely in shear by severing the concrete between the stirrups, without in any manner disturbing the stirrups. Hundreds of beams in buildings which collapsed have failed in exactly this manner. A beam in which the main reinforcing rods are bent up and anchored beyond the edge of support cannot possibly fail without either severing the rod completely or pulling it out of the concrete in which it is fully anchored. There could be no more positive reinforcement than this—and this is the type of reinforcement which Mr. Myers says should not be considered, except on rare occasions.

Many designers are using this type of reinforcement for shear, and it is safe to say that there have been no cases of beams that have fallen away from their supports where this type of reinforcement has been used.

The analysis of the stress in bent-up and anchored rods is quite simple. It is the old principle of the hog-rod or the queen-post truss. The shear of the beam for which reinforcement is needed acts as a vertical force, and the component in the direction of the rod is the stress in that rod for which its section and anchorage should be proportioned.

The rods should not be bent down at different distances from the support, but the downward bend should be about at the edge of support. The upward bend should be where the shear is small enough to be taken by the concrete at a unit stress of 50 to 60 lb. per sq. in. If there are concentrated loads, as of beams carried by a girder, the bend of the rod should be under the girder.

Steep bends in main rods and bends at different distances from the support cannot be defended or analyzed. The rods should be bent up in one plane if more than one rod is needed for the shear, and the slope of the rod should be a flat one. This avoids sharp bends; for no reinforcing rods should be under high stress at the bends, if these bends are sharp.

The foregoing rules for designing shear reinforcement would supplant all rules for the spacing of stirrups; they are based on a rational and complete analysis of the beam stresses. The system leaves to the concrete the mere function of holding by proper embedment and anchorage the reinforcing steel; and places on the steel the function of sustaining direct, calculable tension, which both the concrete and the steel are capable of and fitted to do.

It is significant to note that the same series of tests, made on existing buildings, which disclosed small compressive stresses in stirrups in beams that were cracked, showed large stress in shear reinforcement consisting of bent-up and anchored main steel rods, the type of shear reinforcement that Mr. Myers would reject; and no cracks could be found in the beams thus reinforced.

Mr. Myers' requirement that the steel reinforcement be designed to take all the shear is an unnecessary burden on the design. Concrete is abundantly able to take shear up to amounts of 50 to 60 lb. per sq. in. provided it is safe-

guarded against the start of a failure produced by a crack. If the horizontal reinforcement extends well beyond the edge of support that reinforcement will tie the concrete together on the tension side of the beam and render it capable of carrying the unit shears just named with complete safety.

H. C. SANDBECK,* JUN. AM. SOC. C. E. (by letter).†—It is a general principle that in the ideal design of a structure the same factor of safety applies throughout. For the sake of simplicity, however, the engineer often is obliged to build certain parts with excess strength. He either "figures on the safe side" to simplify calculation, or he orders superfluous material to simplify erection; but he will never for the sake of simplicity allow a single part of a structure to have a smaller factor of safety than is required for the structure as a whole.

It seems, however, that some of the simplifications recommended by Mr. Myers in calculating reinforcing steel for concrete beams and slabs will lead to designs which have a considerable smaller factor of safety than is generally required for reinforced concrete.

The writer agrees with the "General Principles" advanced by the author‡ that reinforced concrete beams and slabs are non-homogeneous and have a varying moment of inertia. (It is believed, however, that this latter fact can be taken into consideration in the proper use of the elastic theory with only slight error.) Further, it is correct in a majority of cases to consider monolithic built beams and slabs as fixed at their supports. These considerations, however, will not justify a reduction of steel over supports such as Mr. Myers recommends.

For beams and slabs continuous for more than two spans the Joint Committee on Specifications for Concrete and Reinforced Concrete specifies§ that for maximum positive moment near the center and maximum negative moment at the support of interior spans, $M = \frac{1}{12} w\, l^2$. Mr. Myers states‖ that these beams "should be designed for $M = \frac{1}{12} w\, l^2$ at mid-span and $M = \frac{1}{24} w\, l^2$ at the supports."

It is well known that a uniformly loaded beam fixed at both ends and with a constant moment of inertia, produces a bending moment, $M = + \frac{1}{24} w\, l^2$ at the center, and $M = - \frac{1}{12} w\, l^2$ at the supports. Assuming the moment of inertia to have a constant value, I_N, where the bending moment is negative and another constant value, I_P, where the bending moment is positive, it is an easy matter to calculate the moments produced.

This assumption is not quite correct because concrete in tension will add to the stiffness of the beam where it is not cracked (near points of contra-

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† Received by the Secretary, August 19, 1926.

‡ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1106.

§ *Loc. cit.*, October, 1924, Papers and Discussions, p. 1190.

‖ *Loc. cit.*, August, 1926, Papers and Discussions, p. 1112.

flexure); but as the steel is dominant in determining the stiffness, the approximation is believed to be very close.

A uniformly loaded beam, fixed at the supports, has a moment diagram, as shown in Fig. 7 (*a*), and an elastic line, as shown in Fig. 7 (*b*). Any two tangents to the elastic line intersect at an angle, ϕ, such that,

$$\phi = \int \frac{M}{E\,I}\,d\,x$$

the integral being taken between the tangent points.

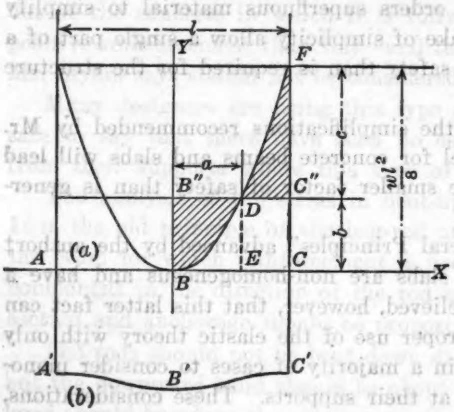

FIG. 7.

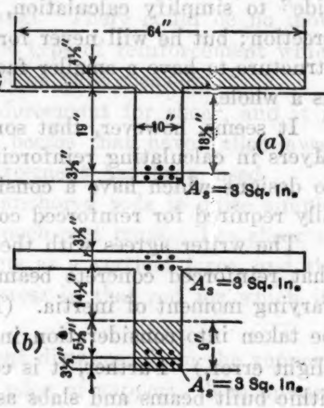

FIG. 8.

On account of symmetry, the tangent to the elastic line at B' is parallel to the tangent at C', therefore, between B and C:

$$\int \frac{M}{E\,I}\,d\,x = 0$$

As $\int M\,d\,x$ is the area of the moment diagram, the positive area, B-B''-D, divided by I_P must equal the negative area, D-C''-F, divided by I_N.

The formula for the parabola in a co-ordinate system, $X = Y$, is $y = k\,x^2$. For $x = \frac{l}{2}$, y is $\frac{w\,l^2}{8}$, and as $\frac{w\,l^2}{8} = k\frac{l^2}{4}$, therefore, $k = \frac{w}{2}$, and, generally, $y = \frac{w}{2}\,x^2$.

The area, B-B''-D, is:

$$\frac{2}{3}\,a\,.\,b = \frac{2}{3}\,a\,.\,\frac{w}{2}\,a^2 = \frac{w}{2}\,.\,\frac{2}{3}\,a^3$$

The area, D-C''-F, is:

$$\int_a^{\frac{l}{2}} y\,d\,x - b\left(\frac{l}{2} - a\right) = \int_a^{\frac{l}{2}} \frac{w}{2}\,x^2\,d\,x - \frac{w}{2}\,a^2\left(\frac{l}{2} - a\right)$$

$$= \frac{w}{2}\left[\frac{l^3}{24} + \frac{2}{3}\,a^3 - \frac{a^2\,l}{2}\right]$$

Then,

$$\frac{1}{I_N}\,.\left[\frac{l^3}{24} + \frac{2}{3}\,a^3 - \frac{a^2\,l}{2}\right] = \frac{1}{I_P}\,.\,\frac{2}{3}\,a^3 \ldots\ldots\ldots\ldots (7)$$

From Equation (7), a, and, therefore, b, can be found when I_P and I_N, or the proportion between them, are known.

It will be seen that if $I_P = I_N$, a^3 cancels, and Equation (7) will be written,

$$a^2 = \frac{1}{12} l^2$$

therefore,

$$b = \frac{w}{2} a^2 = \frac{1}{24} w l^2$$

as is given in all handbooks.

In order to get an idea about how much the moment of inertia changes at the points of contraflexure, consider the beam shown in Fig. 8. This beam is designed for $M = \frac{w l^2}{12} = 1\,000\,000$ in-lb. at both the center and the support. One-half the positive reinforcement is bent up over the supports, and all bars overlap at the supports. The cross-section at the center is shown in Fig. 8 (a) and at the supports in Fig. 8 (b). Neglecting tension in the concrete, the moment of inertia can be computed as follows:

$$I_P = \frac{1}{12} \times 64 \times 4.5^3 + 64 \times 4.5 \times 2.75^2 + 15 \times 3 \times 18.5^2 = 18120 \text{ in.}^4$$

$$I_N = \frac{1}{3} \times 10 \times 9^3 + 15 \times 3\,(5.5^2 + 14.5^2) = 13230 \text{ in.}^4$$

$$\frac{I_N}{I_P} = \frac{13230}{18120} = 0.73$$

If this value is used in Equation (7), the value of a will be found (by trial) to be $a = 0.3l$.

Therefore,

$$b = \frac{w}{2} a^2 = 0.045\, w\, l^2 = \frac{1}{22.2} w\, l^2$$

$$c = 0.125\, w\, l^2 - 0.045\, w\, l^2 = 0.08\, w\, l^2 = \frac{1}{12.5} w\, l^2$$

The design has nearly the desired safety factor over the supports and is about 36% too strong at the center.

If A_s and A_s' in Fig. 8 (b) were reduced to 1.5 sq. in., the neutral axis would move to 7.5 in. from the bottom, and the moment of inertia would be:

$$I_N = \frac{1}{3} \times 10 \times 7.5^3 + 15 \times 1.5 \times (7.5^2 + 16^2) = 8500$$

$$\frac{I_N}{I_P} = \frac{8500}{18120} = 0.468$$

Computed as before, the moments will be

$$M = + \frac{1}{18.2} w l^2$$

and

$$M = - \frac{1}{14.3} w l^2$$

in the middle and over the support, respectively.

The section has a resisting moment:

$$M = f\,s \times p \times j \cdot b\,d^2$$
$$= 16\,000 \times 0.0064 \times 0.90 \times 10 \times 23.5^2 = 500\,000 \text{ in-lb.}$$

The moment previously used was:

$$M = \frac{w\,l^2}{12} = 1\,000\,000 \text{ in-lb.}$$

therefore, this section is good for only one-half as much, $\frac{w\,l^2}{24}$, or just the amount recommended by Mr. Myers. The steel over the supports, however, would be strained to,

$$16\,000 \times \frac{24}{14.3} = 27\,000 \text{ lb. per sq. in.}$$

It seems inadvisable, therefore, to use less than the amount of steel over the supports recommended by the Joint Committee. By decreasing this amount, the negative bending moment is reduced; but the resisting moment of the beam is reduced still more, and the steel is over-stressed.

GRIT CHAMBER PRACTICE

A SYMPOSIUM

Discussion*

By John F. Skinner, M. Am. Soc. C. E.

John F. Skinner,† M. Am. Soc. C. E.—In opening the discussion of the subject so ably handled the speaker will present some facts and observations concerning the design, construction, and operation of the grit chambers at the Rochester, N. Y., plants. These plants will first be briefly described, after which the general subject will be discussed.

Data are available from three disposal plants at Rochester, N. Y., as follows: The Irondequoit Plant, designed by the late Emil Kuichling, M. Am. Soc. C. E., consists of coarse racks with 3-in. openings; six parallel grit chambers, 10 ft. wide, 9 ft. deep, and 90 ft. long, each followed by a 12-ft. Reinsch-Wurl screen (four now installed); influent and effluent channels, and float-regulated gates which admit the screened sewage to twenty Imhoff tanks (ten now installed); eighty sludge beds (forty now installed); 66-in. lock-bar steel outlet pipe, 9 000 ft. long, extending 7 000 ft. into Lake Ontario in 50 ft. of water; at present receiving about 36 500 000 gal. per day of combined sewage from 275 000 persons.

The Brighton Plant, designed by the speaker, consists of three parallel grit chambers, each 4 ft. wide, with a deep pocket; fine racks with $\frac{11}{16}$-in. openings; penstock; two Imhoff tanks (one now installed); 2 acres of sprinkling filters (1 acre now installed); four sludge beds (two now installed); four final sedimentation basins (two now installed); and two 12-in. outlet pipes, 1 100 ft. long, into Irondequoit Creek; at present receiving about 1 500 000 gal. per day of sanitary sewage from 10 000 persons.

The Charlotte Plant, designed by the speaker, consists of two parallel grit chambers, each 4 ft. wide and 40 ft. long, with an end pocket; fine racks, with $\frac{11}{16}$-in. openings; 6 and 7-in. air-lifts; circular Imhoff tank; sludge beds; and $\frac{12}{17}$-in. outlet pipe, 300 ft. long, to the Genesee River; operating house with electrically operated air compressors in triplicate; at present receiving from 300 000 to 1 000 000 gal. per day of combined sewage from a population of from 3 000 to 20 000, as the plant is in a suburb which includes the city's principal summer resort. Fig. 8 (*a*) to (*d*) shows sections of the grit chambers of

* This discussion (of the Symposium presented at the meeting of the Sanitary Engineering Division, Montreal, Que., Canada, October 15, 1925, and published in September, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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the plants mentioned, together with a cross-section of the grit chamber of the proposed Maplewood Plant. Available records from these plants will be used for illustration.

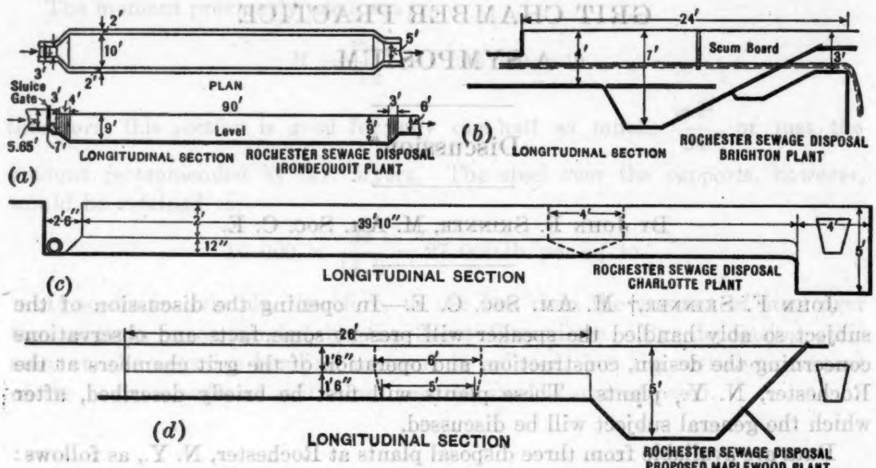

FIG. 8.—SECTIONS OF GRIT CHAMBERS OF SEWAGE DISPOSAL PLANTS, ROCHESTER, N. Y.

I.—Method of Approach.—At the outset the following should be considered:

(*A*) Sources of grit in sewage: (1) In sanitary sewage, grit may come from the bathing of people and washing of animals; from erosives and erosive cleansers; from the washing of cellars and floors which contain dust and earth carried in by the feet and clinging to commodities; from the washing of vegetables and food stuffs and the preparation of food; from silt which enters the sewers through openings and poor joints and with ground-water from the surreptitious drainage of trenches and excavations into the sewers; (2) in storm water, grit may come from street wash; from dust blown on roofs; from erosives; and from the draining of excavations; and (3) in combined sewers and storm overflows all the sources just mentioned may contribute grit.

(*B*) Character of grit: Physical and chemical properties. Properties such as organic content, fineness, and specific gravity are exhibited in Table 5.

Of the clean grit from the Irondequoit grit chamber, 1 cu. ft. drained and packed weighs 85 lb. The specific gravity of the mass is 1.36. With voids assumed at 35%, the average density of the material will be 2.08, as shown in Table 5 (*A*) for mixed samples. This grit, although from a combined sewer system, shows the same berry and grape (raisin) seeds that are found in large quantities in the Imhoff sludge at the Brighton Plant which treats the sewage from a separate system. These seeds, together with particles of coal, are the principal objects which give "loss on ignition". Neither the seeds nor the larger particles of coal pass the 20-mesh sieve, which fact readily accounts for the 37% loss on ignition above this size and only 15% below it. It will be further noted that the specific gravity of the finer material is 2.23, whereas that held on the 20-mesh sieve is 1.45.

Only a little of this material is putrescible, but at present considerable quantities of grain are found in the grit, the probable source being the wastes from home-brewing processes. This material is found in layers near the downstream end of the deposit in the Irondequoit grit chambers and at about one-half the depth of the deposit. A few days after de-watering the chamber, in warm weather, this material is foul smelling when uncovered.

TABLE 5.—MECHANICAL ANALYSIS OF GRIT FROM DETRITUS CHAMBERS, IRONDEQUOIT PLANT, ROCHESTER, NEW YORK.

(*A*) JUNE 3, 1919.

Material.	Specific gravity.	Loss on ignition, percentage.	Mesh per linear inch.	WEIGHT, IN GRAMMES.	
				Passing each sieve, retained on next.	Total amount passing.
Mixed samples........	2.08	23.3	4	33.20	190.18
Held on 20-mesh sieve..	1.45	37.0	10	38.95	156.98
Passing 20-mesh sieve..	2.23	15.0	20	30.36	118.03
....	....	....	30	30.42	87.67
....	....	....	40	30.20	57.25
....	....	....	50	9.63	27.05
....	....	....	80	6.65	17.42
....	....	....	100	10.77	10.77
Total.............	....	....	....	190.18	190.18

(*B*) NOVEMBER, 1921, AND JANUARY, 1922.

Material.	Specific gravity.	Loss on ignition, percentage.	Mesh per linear inch.	Passing each sieve, retained on next.	Total amount passing.
Mixed samples, Nov. 28, 1921, and Jan. 30. 1922..........	1.77	....	4	32.2	160.3
....	....	....	10	40.5	128.1
....	....	....	20	27.5	87.6
....	....	....	30	16.5	60.1
....	....	....	40	14.5	43.6
....	....	....	50	27.5	29.1
....	....	....	80	0.8	1.6
....	....	....	100	0.8	0.8
Total.............	....	....	...	160.3	160.3

(*C*) Quantity of grit: Certain substances and objects will be deposited in grit chambers at low velocity, and at slightly higher velocity will be carried on and caught on the screens. The total of such material at the three plants is shown in Table 6.

The Irondequoit Plant began operation in March, 1917, the Brighton Plant on March 1, 1916, and the Charlotte Plant in November, 1921.

It will be noted that wide variations in the quantity of grit collected occur from year to year, due to the fact that most of this material is brought down when the velocity in the sewers is increased by rain and melting snow. It, therefore, varies with the meteorological conditions, as they produce greater or less scouring velocities.

The quantity of grit deposited is commonly given as so many cubic feet, cubic yards, or pounds per million gallons of sewage. This may be a convenient statement, but in determining the capacity for grit in a proposed design it must be remembered that this material does not come to the plant with each 1 000 000 gal. of sewage, but that it settles and is stored in the sewers (if their dry-weather velocity is small) until a storm flow occurs to wash it to the treatment plant. Even if the sewers have self-cleansing velocities at all times, the great mass of grit is washed in from the streets during storms and reaches the plant with the storm flow. The capacity for storage of grit, therefore, must be based on the quantity that may be brought down by the maximum of storms which may occur in the interval between assumed reasonable periods of cleaning.

TABLE 6.—QUANTITIES OF GRIT AND SCREENINGS REMOVED FROM THE ROCHESTER, N. Y., DISPOSAL PLANTS, IN CUBIC FEET PER MILLION GALLONS.

Year.	Screenings:			Grit:		
	Irondequoit.	Brighton.	Charlotte.	Irondequoit.	Brighton.	Charlotte.
1916	....	0.86*	....	....	4.29*	....
1917	4.26*	0.66	....	1.99*	0.73	....
1918	6.11	0.42	....	1.93	....	....
1919	6.55	0.36	....	2.67	....	....
1920	4.58	....	....	2.58	....	....
1921	3.60	2.77	....	2.40	....	....
1922	3.94	3.88	....	3.22	....	....
1923	5.13	5.93	3.12	3.07	....	....
1924	5.15	4.98	3.13	2.56	....	....
1925	5.35†	5.36†	2.27†	3.02†	....	....
Weighted mean...	4.97	2.81	2.89	2.60	....	....

* 10 months.
† 9 months.

(*D*) Deposition of grit: sedimentation, hydraulic value, and critical velocities.

Data collected by Allen Hazen, M. Am. Soc. C. E.,* have been used by the speaker in studying several proposed designs for grit chambers. These data, somewhat amplified, together with certain derived information, are given in Table 7, exhibiting the sedimenting characteristics of grit.

Table 7 may be used to determine a tentative length of grit chamber which will allow settlement of the various sizes by multiplying the corresponding periods in Column (6) by the proposed depth of the chamber. This is based on a velocity of 1 ft. per sec. As the experiments on which Table 7 was based were made in still water it is apparent that, in a flowing stream, eddies will occur which will lengthen the path of a particle of grit and thus increase its velocity.

Another disturbing element is the commotion incident upon entry to a chamber through a gate or along a channel which approaches at an angle or

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 63.

changes in direction. In the large grit chambers at the Irondequoit Plant the deposit commences 15 or 20 ft. along the 90-ft. chamber from the entry end. A velocity of 1 ft. per sec. is satisfactory in classifying material and in causing grit to settle and allowing most of the organic solids to pass on. Particles of relatively large size will also settle in the chamber even if their specific gravity is considerably less than the 2.65 mentioned in Table 7.

TABLE 7.—SEDIMENTING CHARACTERISTICS OF GRIT.

Diameter of particle, in millimeters.	Hydraulic Value of Particle at 50° Fahr.		Weight of sand particles of specific gravity 2.65, in grammes.	Number of particles of sand per gramme.	Time required to settle 1 ft., in seconds.
	Millimeters per second.	Feet per second.			
(1)	(2)	(3)	(4)	(5)	(6)
1.00	100	0.328	0.001387	720	3.0
0.80	83	0.272	0.000710	1 408	3.7
0.60	63	0.207	0.000299	3 742	4.8
0.50	53	0.174	0.000173	5 882	5.7
0.40	42	0.138	0.000089	11 262	7.2
0.30	31	0.105	0.000037	26 684	9.5
0.20	21	0.069	0.000011	90 090	14.5
0.15	15	0.049	0.0000047	213 675	20.4
0.10	8	0.026	0.0000014	724 652	38.4
+0.095	7.6	0.025	0.0000012	......	40.0

For large plants where, say, 20 000 000 gal. daily may pass through a single chamber, a width of 10 ft. and a depth of 4 ft. will allow 1 ft. of deposit and still maintain a velocity of 1 ft. per sec. Broad tanks require less depth to maintain a given velocity and the length of the tank is proportional to the depth for a given cross-section. The tank must not be too broad for convenient cleaning, nor for the maintenance of uniform velocity in the cross-section.

For small plants where 2 000 000 to 3 000 000 gal. daily will pass through the chamber, a channel 4 to 6 ft. wide, and with the depth varied by a suitably placed weir, either fixed or adjustable, will readily maintain a desirable velocity. In yet smaller plants, more uniform velocity can be maintained in a chamber 4 ft. wide, with a V-shaped bottom formed with slopes of 1 on 2. In such plants a deeper pocket at the end of a uniform channel is a valuable feature, for deposits of coarser grit in the up-stream sections of the channel may be washed down by a hose stream into the pocket, thus maintaining a relatively clean channel of constant cross-section and, at the same time, collecting the grit at one point for removal.

In all cases, dual or multiple chambers are desirable, both for convenience in cleaning, efficiency, and economy, because in storm flows two or more may be put into service thus maintaining a proper velocity.

For a large chamber like those at the Irondequoit Plant, the grit begins to build up 15 or 20 ft. from the up-stream end, and as this bar of grit increases in height and reduces the cross-section of the stream, the finer grit is carried along at the higher velocity and deposited farther down stream after it has had time to reach the deeper bottom.

In general, the coarser material is deposited up stream, but fine material is always found along the bottom beneath the coarse material in the downstream half of the chambers. Probably as the detritus collects in the channel more and more of the finer material is carried along into the Imhoff tanks by the increased velocity. Indeed, at the Irondequoit Plant, the noticeable presence of grit on the brushes of the Reinsch-Wurl screens is one of the symptoms which indicate when the grit chambers require cleaning.

A study of critical velocities at which various sized particles will move or be deposited would be interesting but, in general, the grit to be removed is so much coarser than that which is entangled in organic solids and so much heavier than the organic solids that essential classification may be effected if a velocity of about 1 ft. per sec. is maintained for a sufficient period.

(*E*) Grit chambers are desirable:

1.—Under various conditions, such as: (*a*) At all times in treatment plants receiving sewage from a combined system. If, by means of grit chambers, 4 to 6 cu. ft. of grit per 1 000 000 gal. can be removed from combined sewage, the inclusion of grit chambers is justified in the design of a plant; (*b*) in plants treating sanitary sewage where there is danger that the heavy solids may clog the sludge pipes of deep tanks. Some grit will of necessity go on into the Imhoff tanks, but any large quantity of heavy sand will render the drawing of sludge difficult; (*c*) in plants where mechanical devices such as pumps, water wheels, and fine screens cleaned by brushes are liable to excessive wear if the grit is not removed; (*d*) where a long outlet pipe discharges the sewage or effluent at a low velocity and there is consequently a liability of sedimentation in the pipe and reduction of its capacity; (*e*) at storm-water overflows,* where it becomes necessary to reduce the silting up and pollution of streams from this source.

2.—To remove coarse and heavy material as fine as economically advisable with the minimum of putrescible organic matter. A small quantity of grit necessarily continues on through the preliminary processes, entangled or enveloped in organic solids from which it becomes disengaged in the process of digestion in the Imhoff tanks. The quantity and size of this material have been determined in three instances as shown in Table 8.

The quantity of so-called grit in the samples of Imhoff sludge from the Irondequoit Plant varies, as exhibited in Table 8, from 13.8% of the solids (dry basis) at the first drawing in the spring to 3.8% for the drawing in midsummer. This "grit" is to some extent made up of seeds and coal, as is indicated by the "Loss on ignition". This loss is greater in the larger sizes, the same as in grit-chamber material (Table 5).

The Irondequoit samples indicate that after ignition only 10% of the grit from the grit chambers is below the "passing 50, retained on 80-mesh" size, whereas in the "grit" from the Imhoff sludge the same size amounts to 65% of residue after ignition. This gives a practical limiting size of 0.20 mm. which, if it has a specific gravity of 2.65, may be expected to settle 1 ft.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 438.

in about 14.5 sec. in still water. The grit that passes after such removal has been found to cause no difficulty in an Imhoff tank and can be discharged readily with the sludge.

TABLE 8.—MECHANICAL ANALYSIS OF GRIT FROM IMHOFF TANK SLUDGE.

(A).—BRIGHTON PLANT, 1917 (300 GRAMMES OF WET SLUDGE).

Characteristics of Wire Mesh Sieves.					Weight, in Grammes.	
Mesh per linear inch.	Diameter of wire, in millimeters.	Side of Square Opening. Millimeters.	Side of Square Opening. Inches.	Side of Square Opening. Fraction.	Passing each sieve, retained on next.	Total quantity passing.
4	.....	.....	....	....	1.06	2.46
10	.....	.....	....	....	0.31	1.40
20	.....	.....	....	....	0.10	1.09
30	.....	.....	....	....	0.07	0.99
40	.....	.....	....	....	0.19	0.92
50	0.212	0.2960	0.01165	1/86	0.28	0.74
80	0.140	0.1775	0.00699	1/148	0.24	0.45
100	0.099	0.1550	0.00610	1/164	0.19	0.22
200	0.051	0.0760	0.00299	1/384	0.03	0.03
Total	......	......	....	....	2.47	....

(B).—IRONDEQUOIT PLANT, 1922.

First Drawing of Sludge in 1922. 60 Grammes of Dried Sludge Yielded 8.3 Grammes of Heavy Solids, "Grit."				Drawing of Sludge, July 7, 1922. 138.60 Grammes of Dried Sludge Yielded 5.2 Grammes of "Grit."	
	Weight, in Grammes.			Weight, in Grammes.	
Mesh per linear inch.	Passing each, retained on next.	Total passing.	Loss on ignition.	Passing each, retained on next.	Total passing.
4	0.0	8.3	...	0.6	5.2
10	0.4	8.3	0.4	1.0	4.6
20	0.4	7.9	0.3	1.0	3.6
30	0.5	7.5	0.5	0.4	2.6
40	0.6	7.0	0.4	0.4	2.2
50	4.4	6.4	2.9	1.1	1.8
80	0.3	2.0	−0.1*	0.2	0.7
100	1.7	1.7	0.1	0.4	0.5
200	0.0	0.0	−1.4*	0.1	0.1
Total........	8.3	8.3	3.1	5.2	5.2

* Larger sizes broken down by burning caused an increase in weight of these sizes.

II.—Design of Grit Chambers.—

(*A*) Conditions of service:

1.—If a territory is thoroughly built up and the population fairly constant the conditions will be relatively uniform at the plant and the design may be definitely fixed.

2.—If the territory is new and increasing in population the engineer must discount the future in his design, building a number of units or providing for their construction later.

3.—If the territory presents a widely varying population, such as a summer resort, with a large number of cottagers and crowds of transients on pleasant days, these wide variations must be considered in the design for it will be desirable to maintain an approximately uniform velocity through the grit chambers at all times.

(*B*) Type of chamber:

1.—Long chambers of uniform section are economical to construct and may be laid out in parallel, with common partition walls. They are convenient to clean and may be spanned by travelers running on rails on or above the longitudinal walls.

2.—Short chambers with deep pockets have the advantage of collecting grit at one point for removal, the mean velocity in the pocket and in the channel of approach being inversely as the depth of water. Such pockets call for careful design to avoid, on the one hand, eddying and scouring and, on the other, a stratum of dead water in the bottom of the pocket over which the sewage may flow from the channel of approach without appreciable change of velocity. In the latter case the channel would have to be designed as a long one, using the pocket merely for storage of the deposited grit. This uniform velocity may be maintained over the pocket by a false bottom of slats through which the deposited grit may fall.

3.—(*a*) Long chambers of increasing cross-section are suggested as a type which may be useful where it is desired to collect fine grit. This form will assist in classifying the material for larger heavy particles will settle near the entry end and the finer, progressively along the channel as the velocity decreases. The depth should be constant and the channel become wider down stream; (*b*) tanks of uniform width, but with greatest depth near the entry end, growing shallower down stream, usually have a reverse current along the bottom for some distance from the deep end. This serves to move the fine grit which has settled in the shallow end back to the deep section.

4.—The position of entry gates and coarse racks and the means of baffling at entry so that the stream will smooth out as soon as possible are important, as well as a convenience in making repairs or replacements. A type of gate which will admit water over the full width of the chamber is preferable to a small gate. Coarse racks placed in each chamber a short distance up or down stream from the gate may be inclined for convenience in raking and if up stream from the gates, the entire rack may be slid down grooves in the side walls so as to be readily removed for repairs. If down stream from the entry gate it will be exposed when the chamber is dewatered, and may act as a baffle. Baffling to break up a rapid thread in the stream can best be accomplished by trial.

The deposit in the Irondequoit grit chamber commences 15 or 20 ft. down stream from the inlet. It builds up to about 3 ft. depth decreasing slightly down stream. In September, 1925, when preparing to discuss this subject,

the speaker suggested that a 12-in. plank baffle with its top inclined down stream be set across the bottom of one of the chambers at its up-stream end, the purpose being to deflect the flow upward, obviating the scour on the bottom, and causing an eddy in which deposit might occur. The baffle was placed in No. 2 Chamber, which was operated in parallel with No. 3 Chamber and under identical conditions during a period including a week of heavy rains. When the chambers were dewatered the utility of the baffle was at once apparent. The deposited grit started at the baffle and even gathered in the corners up stream from it where formerly the bottom had been scoured clean. The operator at the plant further reports that 116 cu. yd. of grit were removed from this chamber, whereas the usual removal is from 66 to 70 cu. yd. One previous instance only gave a removal of 96 cu. yd. This simple baffle, therefore, has added about 50% to the efficiency of the grit chamber.

III.—Location of Grit Chamber in Flow Line of Treatment Plant.—The grit chamber should be up stream from fine racks or screens, for the reason that the heavy grit will settle out at a relatively high velocity and thereby relieve the sewage before subsequent treatment wherein the grit might cause damage or difficulties.

IV.—Methods of Removal of Grit.—

(*A*) Small plants handling sanitary sewage only will produce but a small quantity of grit, which can be shoveled into pails. When this quantity has become somewhat greater, steel cans 17 in. in diameter and 24 in. deep, with bales, can be efficiently handled with a chain tackle and sidewalk ash hoist. This hand work requires de-watering of the chamber.

(*B*) In large plants, power may be used with dipper or clamshell buckets, or bucket and belt, or screw conveyors. These may be used after stopping the flow through the chamber, but do not necessarily require de-watering. Clamshell and dipper buckets are likely to injure the bottom masonry. For this reason the clamshell formerly used at the Irondequoit Plant has been discarded and the body of a Koppel Industrial Railway car is lifted from its truck and lowered into the chamber by the shovel. After filling by hand the car body is replaced on its truck and when a train is made up, a storage battery locomotive hauls it away. This requires de-watering of the chamber, which in a few days drains satisfactorily.

At Rochester the electric power is generated hydraulically from the fall of the sewage to the level of Lake Ontario. About 150 h.p. is available at all times. In Syracuse, N. Y., clamshell buckets the width of the chamber ride on rails flush with the bottom of the chamber and scrape the bottom clean. In small municipal plants where there is usually considerable hand labor available, an expensive mechanical equipment requiring a skilled operator, as well as repairs and maintenance, will often not justify its cost.

In a large plant, a bucket and belt conveyor running on tracks on the longitudinal walls of the chambers and provided with a transfer table offers an attractive solution. A sand sucker and a vacuum ejector offer interesting methods for investigation.

(*C*) Continuous removal of grit has not been generally practical as the grit is not deposited uniformly or continuously. Continuously moving

scrapers connected by linked chains running longitudinally and rising on a ramp at one end of the chamber have been used, and a curved or semi-circular cross-section for the bottom of the chamber with scrapers operating transversely has been suggested.

V.—Final Disposition of Grit.—Grit is generally used for filling and is often in demand. Due to its porosity it is valuable for mixing with soil growing certain garden crops, notably, cucumbers, squash, melons, and tomatoes. It is also useful as a top dressing for dirt or gravel paths and drives for light or occasional traffic. In this respect, it is similar to coal cinders.

UNIT STRESSES IN STRUCTURAL MATERIALS

A SYMPOSIUM

Discussion*

By Messrs. Lee H. Miller and E. F. Kenney

Lee H. Miller,† M. Am. Soc. C. E. (by letter).‡—Even if the Engineering Profession were to agree on a basic unit stress for structural steel, such agreement would constitute only the first of many steps necessary in establishing fair competitive conditions on a uniform basis. Fair competition depends on clear-cut specifications and rulings which are mutually understood by, and acceptable to, both buyers and sellers. Obviously, the establishing of a unit stress can not accomplish this condition any more than fixing the dimensions and weight of a football will assure fair competitive conditions in intercollegiate contests. Public confidence is based on a general belief that the conditions are fair, and that they are understood by all concerned, whether they be contestants or spectators.

The engineer resembles a referee or umpire, in that he assumes the obligation between buyer and seller of maintaining mutual understanding as to what is expected from each. His position is difficult even under favorable conditions, and with both buyer and seller of responsible standing. Under existing conditions, there is a wide variation in local practice, and no uniformly accepted standards. This often makes his position resemble that of a referee in equity who has no accepted precedents to guide his rulings, with the result that neither buyer nor seller is satisfied.

If the engineer is assumed to be the representative of the buyer, and not a third party functioning between buyer and seller, there are two major interests concerned in promoting better conditions for both. Responsible engineers and responsible sellers of structural steel have long felt the necessity of some step being taken, and that sooner or later this necessity would be crystallized into action.

Responsible sellers with huge capital invested in plant equipment have been confronted with financial losses due to their unwillingness to meet the constant lowering of standards which past conditions have encouraged, both in the engineering and manufacturing fields. Through the American Institute of Steel Construction these sellers have initiated, with the co-operation of

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‡ Received by the Secretary, January 4, 1926.

prominent engineers, the promotion of a uniform specification and code of practice which together provide definitions relating to all the points of interest between buyer and seller.

A committee of five men of high standing in the architectural, engineering, and academic professions, was asked to prepare a standard specification. These men have no financial connection with the sellers of structural steel. Four of them stand high in the Engineering Profession; the fifth is a senior member of a large architectural firm in the Middle West. They agreed unanimously that a unit stress without a specification requiring its use in accordance with good engineering practice, would be of no value, and they have produced what many leading engineers have approved and accepted as the best existing specification on this subject.

Their original discussion considered a specification on a basic unit stress of 20 000 lb., but as a representative of the manufacturing interests, the writer advised them that the fabricating industry was more interested in uniform practice than high stresses, and requested that the basic stress should not exceed 18 000 lb.

In the two years following the publication of this specification, about sixty cities in the United States and Canada have permitted its use for building code regulations and many buyers have required its definitions as the basis of their purchases.

Outside the conditions fixed by this specification, there are many things on which no uniform understanding existed between buyer and seller, and which were the occasion of constant controversy. These have been exhaustively listed, and after careful analysis, rulings have been fixed in a standard code of practice, which deals exclusively with physical conditions and does not touch those which are ethical.

Definitions that are unfair to either buyer or seller cannot survive, and for this reason the co-operation of many engineers and architects was sought in developing the various rulings of the code. The correction of economic evils resembles the treatment of physical diseases in that a proper diagnosis of the conditions and causes is of primary necessity. The manufacturers who are daily receiving plans and preparing prices on work from engineers in all sections of the country, are forced into more intimate contact with the variable conditions than the engineers or architects. They have been obliged to employ expensive and efficient talent to study all the conditions and requirements of plans and specifications in order to detect unusual or difficult obligations. Manufacturing conditions are seldom the same as fixed by two engineers, and constant juggling is necessary to harmonize them with shop practice, all of which costs the buyer money or is absorbed as a loss by the seller. There are many tricks of evasion of building code requirements by both incompetent and competent engineers, neither of which observes the 16 000 lb. stress. The incompetent designer knows there is a wide margin of safety in the 16 000 lb. stress to cover his incompetence and ignorance, and sees no reason why he should employ a competent engineer when the unit stress is accepted as a proper substitute for technical training and experience. His designs include

nothing that indicates consideration of unit stresses or engineering principles as the material specified is as often wasteful as it is too light.

If an increase in the basic unit stress would lead to a change in this type of design, it would be the first evidence indicating any consideration of stresses, and would have to be accomplished by turning the work over to a competent designer where it properly belongs. This result is already apparent in cities where the 18 000 lb. stress basis is used as building code requirements.

At the Annual Meeting of the Building Officials' Conference held in April, 1925, the Building Commissioner of Boston, Mass., stated that prior to the adoption of the 18 000 lb. stress, practically all plans ignored engineering principles, and that the change had resulted in almost complete elimination of errors. Fabricators have also noticed that plans are being prepared more completely, and that competition is on a more even basis.

The evasions by competent engineers are very different in that they are based on judgment and engineering experience, and although they may not be structurally dangerous, they are unfair competitive practices which encourage a constant lowering of standards. The evasive practices used by competent designers although safe, still constitute as great, if not a greater menace to fair competition than those of the incompetent because they establish precedents for more dangerous evasions. The competent engineer justifies these practices as necessary under existing conditions, as he knows the incompetent designer pays no attention to unit stresses, and the competitive competent designer indulges in similar evasions. The result is that the buyer of structural steel is in the dark as to whether he is purchasing a unit stress, a column formula, an assumption, or an omission.

The question of a technical justification for increased unit stresses has been exhaustively discussed, and it seems almost generally accepted that if properly designed, manufactured, and erected, the material may be used with safety almost to its elastic limit. Such differences of opinion as exist are based not on the ability of the material to function properly, but on the ability of the engineer to design and the manufacturer to fabricate and erect it.

It was not so long ago that a modification of the unit stress was used to take care of the difference between live and dead loads, but the Engineering Profession has practically abandoned this method of disguising its lack of knowledge regarding live load conditions. Competent engineers still express doubt as to their competitor's ability to use higher unit stresses. The answer again is not the unit stress, but the engineering principles which fix the conditions under which the unit stress will be used. The entire question is almost an emergency of the present generation brought about by the fact that there has been greater industrial and economic changes in the 35 or 40 years in which structural steel has been a commercial product than any previous 2 000 years of history.

The transition from the Iron Age to the Steel Age which was the result of temperatures that made possible the commercial production of steel ingots, started a new era, and we stand so close to the threshold that it is difficult to comprehend its significance. During such a transition the development of

individualism and pioneering is natural, but the sooner everybody co-operates in the promotion of fair conditions, the quicker fair competitive conditions will be attained. This has a phase which is of National importance in that under present world conditions, National existence is dependent on ability to mobilize industries promptly more than it is on ability to mobilize man power.

To establish nationally accepted standards, needs more than the mere recommendation of nationally recognized technical organizations. It requires the aggressive effort and co-operation of responsible engineers and manufacturers to the end that the purchaser of structural steel may know that a National standard exists, which provides ample assurance that his structures will be economical and safe.

With all the practices that have come into existence under the 16 000 lb. stress, it is impossible to establish a standard on this basis, particularly when practically the entire Engineering Profession realizes that it does not provide the economy which good engineering and higher unit stresses justify. Engineers must be willing to submerge their individualism and forget that personal liberty entitles each engineer to his private column formula. Standard practice can not be attained as long as the conditions existing in connection with the 16 000 lb. stress continue; and they will continue unless they are made obsolete by the necessity of incorporating engineering principles to replace unit stresses which do not require them.

The interests of the responsible engineer are identical with those of the responsible manufacturer, and co-operation is the most effective way to attain a common objective. The writer is unaware of the Society having in the past recommended any engineering or trade practices as proper for general acceptance, but, nevertheless, he feels that even the informal co-operation of the Structural Division would be of great value in improving conditions in the Engineering Profession and the industrial field, and to the ultimate purchaser of structural steel.

E. F. KENNEY,* ESQ.—In determining the unit working stresses safely allowable in any engineering material, it is of prime importance to consider carefully the uniformity of the material and the degree of certainty with which its behavior under application of stresses can be predicted. If by computation the stresses which will be produced in an engineering structure can be predicted, and a material is available the properties of which in resisting such stresses can be predicted with certainty, the safety of such a structure is simply one of mathematics, and there is no necessity for economic waste in multiplying the so-called factor of safety. If, on the other hand, the character of the structure is such that direct and certain computation of stresses is not possible, or if the properties of the material used cannot be predicted, a much greater margin must be provided between the allowable working stresses and the ultimate strength of the material.

Structural steel is easily the most certain and uniform of engineering materials. This is due primarily to its being the result of a fusion process producing a material which up to the point where it is poured into the ingot

* Metallurgical Engr., Bethlehem Steel Co., Bethlehem, Pa.

mould is liquid. This liquid, agitated by several pourings, cannot fail to be thoroughly mixed and consequently uniform. This natural advantage is supplemented by the great advancement in the art of steel manufacture as the industry has progressed.

Those who have not been in close touch with the history of steel making have little conception of the changes and refinements which have taken place in, say, the last twenty-five years.

Twenty-five years ago practically all the chemical analyses for carbon, the outstanding element determining the strength of steel, were made by the color method. This while quick was, when compared with present combustion methods, quite inaccurate. The necessity for greater certainty has led to its being supplanted entirely by the combustion method. These improved methods helped not only in the analysis of the finished steel but made it possible to determine, with equal accuracy, the progress of the refinement in the open hearth. The old crude fracture test which in the past was the melter's only guide, has been supplemented by these improved analytical methods which largely remove the doubt and make the open-hearth practice almost a certainty.

Along with these strides on the chemical side, the physical testing of the steel has kept pace. Every heat is so thoroughly tested, and the physical results are so correlated with its chemical analysis, that it is regular practice to make a chemical analysis of a small test ingot cast at the time the melt is teemed, and from this analysis predict within a probable error of 1 000 or 2 000 lb., the physical strength of the plates and shapes rolled from that melt of steel. The certainty and uniformity is such that an entire melt of steel can be safely rolled into a required finished product which would probably be nothing but scrap if the predicted qualities should not be obtained.

To get uniformity in the steel, manufacturers have been driven to the closest scrutiny of the raw materials used. Everything going into the charge is analyzed and tested, and must meet closest requirements. To give an instance of the wonderful work which is being done, the record of one of the large blast furnaces at the Johnstown Plant can be cited. The pig iron was used in the manufacture of steel. All this pig iron was specified within a total range of 0.25% in silicon and had to be less than 0.05% in sulfur. The furnace was "blown in", in October, 1924, and from that date to June, 1925, a period of eight months, did not produce one cast outside the close requirements named. This is illustrative of the care with which the modern steel industry is working. These features of steel making have been briefly cited so as to show the precision which is required and attained, and the manufacture is being checked by physical testing which is growing in scope as the years go on. To some the commercial testing of structural steels appears to be just a mechanical gesture. Test after test and melt after melt is tested in the physical testing rooms. All day long at a large mill this goes on. It has gone on for years, and one hears sometimes a complaint that "none of the tests ever fails". They seldom do fail. This is the strongest proof of the certainty and uniformity of structural steel. These words "certainty" and "uniformity" are stressed because they belong peculiarly to structural steels, and they are also the words which justify a relatively high permissible unit stress in design. The

present unit stress is largely due to the desire to take care of the growing loads of railroad rolling equipment. This feature should be considered in fixing unit stresses for railroad bridges, but it should not militate against the use of a proper unit stress in the economic design of other structures.

Some of the previous speakers have advocated the use of yield point in determining the permissible unit stresses. Yield point is not easily determined commercially. Assuming that the method used is the "drop of the beam", the results obtained are a function of the speed of the pulling screws, and also of the speed at which the operator moves his poise. If the method of determination is by plotting the elastic curve, it will be found that these curves are not similar, even in the same structural shape. The necessary operation of straightening affects the elastic curve. The ultimate strength is not so affected. The point of plastic yield is theoretically of interest, but the difficulties in the way of its commercial determination should be carefully considered.

FINAL REPORT OF THE SPECIAL COMMITTEE ON IMPACT IN HIGHWAY BRIDGES

Discussion*

BY F. O. DUFOUR, M. AM. SOC. C. E.

F. O. DUFOUR,† M. AM. SOC. C. E. (by letter).‡—The valuable work by the Special Committee on Impact in Highway Bridges leaves much to be done since it has attached the problem of the floor system only. The report§ is not a final one on impact in highway bridges; and the Committee should be continued in order to give it the opportunity to investigate impact in truss members. Experiments were made by the writer during 1908 to 1913 under the auspices of the University of Illinois and the Illinois State Highway Commission, A. N. Johnson, M. Am. Soc. C. E., State Highway Engineer, and have been reported in part.‖ The results show the importance of requiring the Committee to investigate impact stresses in truss members.

The results obtained by the Committee from its work on floor systems agrees with those of the writer's experiments as far as the distribution of the load on the stringers is concerned and the amount of impact under normal road surface conditions. On stringers of concrete floor bridges the impact allowance should not exceed 40%, while on stringers of plank floor bridges the allowance should not exceed 80 per cent. The high impact percentages obtained with the use of 1 by 2-in. and 2 by 4-in. obstructions are of interest but should not be used in practice as similar conditions do not obtain.

The data here given will, it is hoped, show that the Committee should be continued as the impact coefficients for truss members are entirely different from those for floor systems. The Committee might well consider the probable economy of thickening the concrete floor-slab, thus saving in the floor steel on account of a reduction in the impact.

The writer's experiments were responsible for the Illinois State Highway Commission being the first to omit impact from consideration in designing truss members of bridges with concrete floors, resulting in a considerable saving in the cost of the span. Several other States now follow this method. The amount of the saving in the cost of a span can be appreciated when it is considered that the weight of the floor joists and beams in a truss span is

* This discussion (of the Final Report of the Special Committee on Impact in Highway Bridges, presented at the Annual Meeting, January 20, 1926, and published in March, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Director, Div. of Civ. Eng., Lafayette Coll., Easton, Pa.

‡ Received by the Secretary March 4, 1926.

§ *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 442.

‖ *Journal*, Western Soc. of Engrs., June, 1913.

approximately one-third of the total weight of the steel in the span, and that this one-third has a cost per pound less than that of the remainder.

The instruments used in the experiments noted were the Frankel deflectometer and the Turneaure extensometers. The loads were an 3 800-lb., two-horse team, an 8-ton street roller, a 12-ton farm engine followed by one or more 13-ton stone wagons, a 4 300-lb. automobile, and a 1 700-lb. horse and buggy. With this latter load it was possible to load a bridge to approximately 1 600 lb. per lin. ft., or 100 lb. per sq. ft. of roadway. The speed was not great—about 4 to 6 miles per hour.

It is believed that Figs. 10 to 18, showing the results of these tests, point the way to the necessity of further experiments with the more accurate instruments and methods of the Committee. Each diagram is briefly analyzed in order to call particular attention to what conclusions may reasonably be drawn at this time or possibly in the light of the results of future experiments.

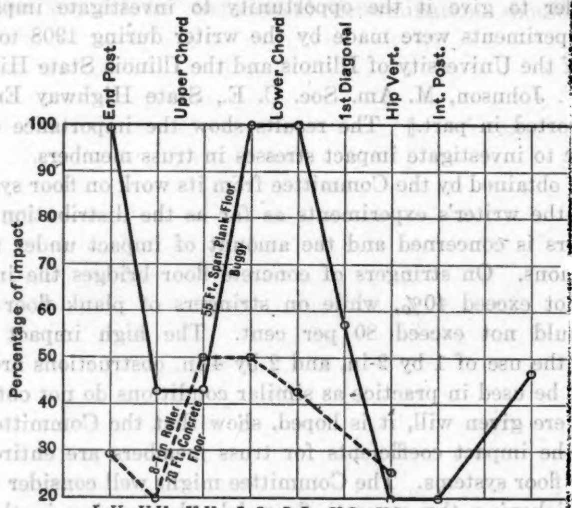

FIG. 10.—VARIATION IN IMPACT FOR DIFFERENT MEMBERS AND DIFFERENT LOADINGS.

Fig. 10 shows that lighter loads produce greater percentages of impact than heavy ones; but it should be appreciated that the sum total of the live load stress plus the impact is greater in the case of heavy loads. As would be expected, the lower chord members, where the load is applied, show greater percentages of impact.

In Fig. 11 the experimental static stress is shown to be much less than the theoretical in almost all the members, especially in the chords in conjunction with a concrete floor. Evidently the concrete floor acts partly as an auxiliary chord and also, functioning as a continuous beam, transfers the load more or less evenly over the entire span.

That the impact stresses are much less than those computed by formulas in common use is shown by Fig. 12, the hip vertical excepted; and this member may be considered in the same class as the floor system.

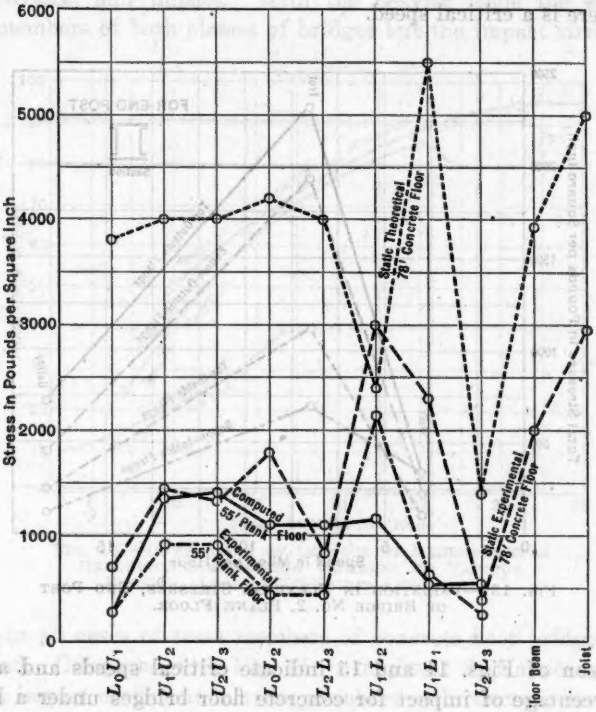

FIG. 11.—RELATION OF THEORETICAL AND EXPERIMENTAL IMPACTS.

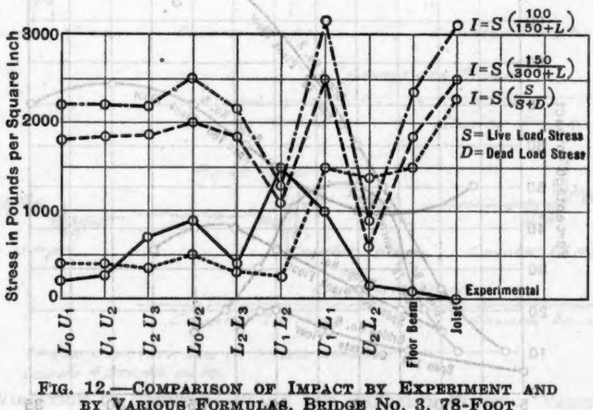

FIG. 12.—COMPARISON OF IMPACT BY EXPERIMENT AND BY VARIOUS FORMULAS, BRIDGE NO. 3, 78-FOOT SPAN, CONCRETE FLOOR, 8-TON ROLLER.

The effect of flexure in a compression member is shown in Fig. 13. It will be noted that the effect of impact continues more or less proportional and that there is a critical speed.

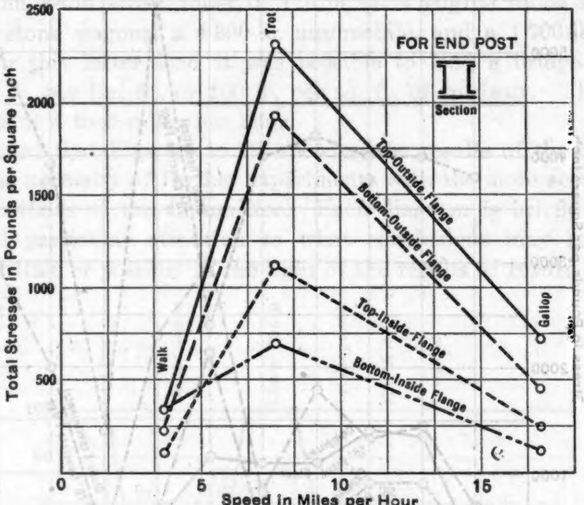

FIG. 13.—VARIATION IN FLEXURAL STRESSES, END POST OF BRIDGE NO. 2, PLANK FLOOR.

Comparison of Figs. 14 and 15 indicate critical speeds and also the fact that the percentage of impact for concrete floor bridges under a light load is not more than 50 per cent. With a heavy load it would be less. Fig. 15 is made up from deflectometer readings at the center of the span.

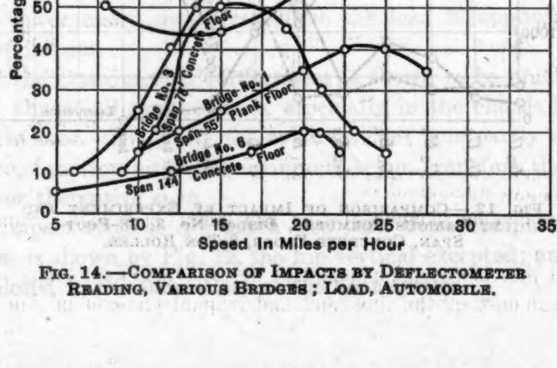

FIG. 14.—COMPARISON OF IMPACTS BY DEFLECTOMETER READING, VARIOUS BRIDGES; LOAD, AUTOMOBILE.

Figs. 16 to 18 show the effect of the concrete floor in reducing vibration, live load, stresses, and impact. With the heavier loads the stresses were greater on members of both classes of bridges but the impact stress was prac-

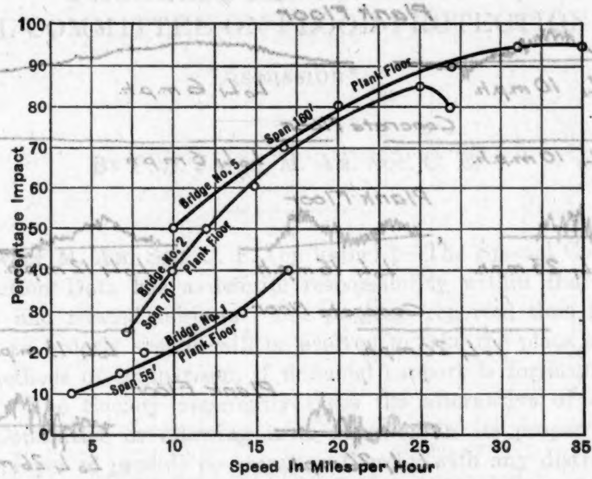

FIG. 15.—COMPARISON OF IMPACTS BY EXTENSOMETER READINGS, COMPRESSION MEMBERS OF VARIOUS BRIDGES. LOAD, 4 300-POUND AUTOMOBILE.

tically zero in all cases of truss members of concrete floor bridges, no matter what the load. Compared with Fig. 15, it might at first appear necessary to allow a 50% impact coefficient in truss members with concrete floors. Figs. 16

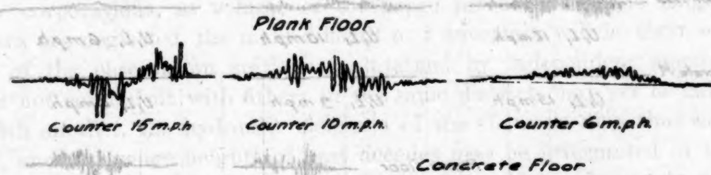

FIG. 16.—VIBRATION OF COUNTER—PLANK AND CONCRETE FLOORS. VARIOUS SPEEDS.

to 18 show that this is not the case, since the concrete floor reduces the live load stress so that the sum of the live load and impact stresses is less in the case of

truss members of concrete floor bridges than the live load stress in the like member of a plank floor bridge.

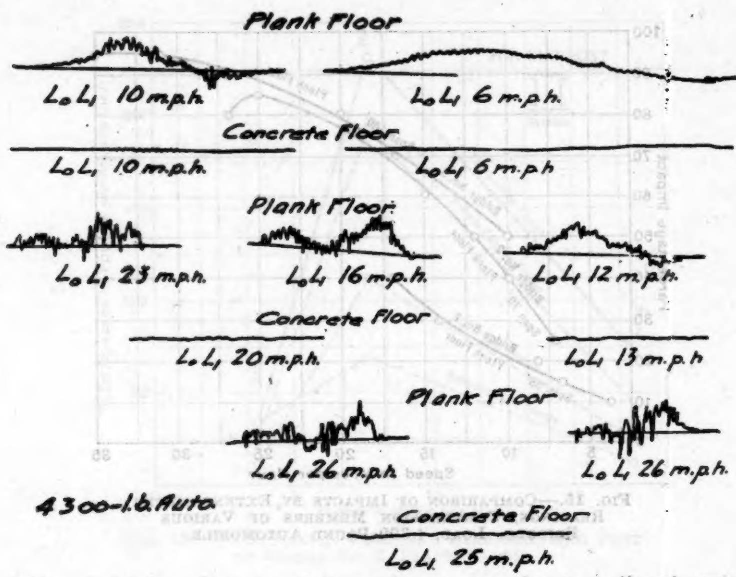

FIG. 17.—VIBRATION OF LOWER CHORD—PLANK AND CONCRETE FLOORS. VARIOUS SPEEDS.

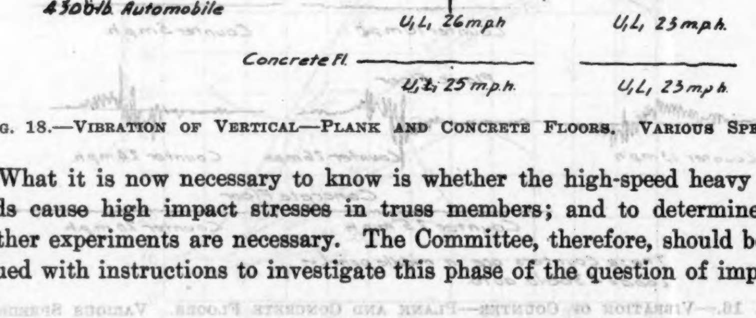

FIG. 18.—VIBRATION OF VERTICAL—PLANK AND CONCRETE FLOORS. VARIOUS SPEEDS.

What it is now necessary to know is whether the high-speed heavy truck loads cause high impact stresses in truss members; and to determine this, further experiments are necessary. The Committee, therefore, should be continued with instructions to investigate this phase of the question of impact.

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON FLOOD-PROTECTION DATA

Discussion*

By C. S. Jarvis, M. Am. Soc. C. E.

C. S. Jarvis,† M. Am. Soc. C. E. (by letter).‡—The Special Committee on Flood Protection Data has a definite responsibility within the entire field of hydrology and related subjects. The progress reported thus far justifies a belief that an orderly system will be evolved to take the place of the usual haphazard methods of comparison, if financial support is forthcoming in the near future. The Society apparently faces the alternative of either abolishing the Committee or allowing it to function in its proper sphere. It would seem proper to provide co-operation thereby with any district that has retained specialists for the study of local flood danger and control. By acting in an advisory capacity, the Committee representative would obtain access to all the basic data collected, and could assist in the process of co-ordination for the local problem and others.

Among the activities which this Committee might well undertake is a thorough correlation of data expressed by the U. S. Weather Bureau in terms of gauge heights, and by the U. S. Geological Survey, various States, and private corporations, as volume of discharge per second. The longer such problems are neglected, the more difficult and uncertain will be their solution. Many of the observation stations maintained by independent agencies, although not coincident with others in the same district, may yet be calibrated one with another, the hydraulic elements of the channels may thus be determined, and the gauge heights of past decades may be interpreted in units of flow. Conversely, any probable rate of run-off would correspond to a known height of the flood-plain, and the zone of inundation would be known and could be vacated in advance instead of during the period of danger.

* This discussion (of the Progress Report of the Special Committee on Flood-Protection Data, presented at the Annual Meeting January 20, 1926, and published in March, 1926, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Care U. S. Bureau of Public Roads, Washington, D. C.

‡ Received by the Secretary, May 15, 1926.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE EDWIN GIFFORD, M. Am. Soc. C. E.*

DIED APRIL 14, 1926.

George Edwin Gifford, the son of Edwin Sands and Harriett (Searles) Gifford, was born at Long Ridge (Stamford), Conn., on March 20, 1864. The family is a Colonial one, his great-grandfather, a native of Rhode Island, having served in the Revolutionary Army. His grandfather was a Baptist minister who held several charges in Eastern New York and Connecticut. His father when a young man taught school, but later engaged in carriage manufacturing and continued in this business until his death.

Mr. Gifford received his early education at Long Ridge, having attended both public and private schools there. He also attended a boarding-school in Noroton, Conn., after which he entered the Stamford High School. For a year or two after graduating from High School, he taught in the country schools near Stamford. In the fall of 1883, he entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1887 with the degree of Civil Engineer.

Immediately after his graduation Mr. Gifford was employed by the Louisville Bridge and Iron Company, of Louisville, Ky., as a Draftsman and Shop Inspector. In 1888, he accepted a position with the King Bridge Company, of Cleveland, Ohio, and except for a short time was with that Company until 1905. He was located in Cleveland until 1894, as Principal Assistant Engineer in the Designing and Contracting Department, and, later, was in charge of designing and contracting. Among other work, he designed a 312-ft. swing bridge for the United States Government at Oakland, Calif.; a cantilever bridge over the Willamette River, at Albany, Ore.; a preliminary design for the New York Central and Hudson River Railroad Company's four-track swing bridge over the Harlem River, at New York, N. Y., and a swing bridge and viaduct for Tacoma, Wash. He also designed extensions to the plant of the Bridge Company at Cleveland.

In 1894, Mr. Gifford was placed in charge of the New York Office of the Company, and took up his residence at Stamford. The New York office handled all matters pertaining to the sale and erection of bridges and structural work in the Eastern territory. During this period, he was also in charge, for the Contractor, of the erection of the New York Central and Hudson River Railroad Company's four-track swing bridge over the Harlem River, and the Harlem Ship Canal Bridge at Kingsbridge, N. Y.

In 1898, Mr. Gifford was, for a short time, a member of the R. H. Hood Company, engaged in contracting in and near New York City. At this time, he moved from Stamford to Maplewood, N. J. His connection with the

* Memoir prepared by Thomas Earle, M. Am. Soc. C. E.

H. R. Hood Company proving unsatisfactory, he returned to his position with the King Bridge Company, and was placed in charge of the New York Office. He remained in this position until 1905, when he became connected with the firm of Milliken Brothers, of New York, as Contracting Engineer. When this Company went into the hands of Receivers, the latter appointed him Chief Engineer.

In 1909, Mr. Gifford assisted in the organization and became Vice-President of the Bowles-Gifford Company, Contracting Engineers, of New York. In 1911, he found it advisable to interest others in this Company, and his efforts resulted in the formation of its successor, The Jobson-Gifford Company, of which he became Vice-President. His active connection with the management of this Company was of short duration, although he retained his interest in it until his death.

In 1911, Mr. Gifford became Secretary of the Bridge Builders' Society and of the Structural Steel Society, both of which had headquarters in New York. Shortly afterward, when these two associations were merged, becoming the Bridge Builders' and Structural Society, he was appointed as Secretary and retained that position until the dissolution of the Society in 1922. Thereafter, he was not active in his profession, but devoted his time to personal and local municipal affairs.

When he became a resident of Maplewood, N. J., Mr. Gifford identified himself actively with all its local affairs. He was a Director of both the Maplewood Bank and the Maplewood Building and Loan Association from their organization. He was one of the early Presidents of the Maplewood Country Club; was active in church and Masonic affairs, having been Past-Master in his lodge. He served as Tax Assessor of the town for a number of years.

While of a rather retiring disposition, Mr. Gifford was held in high esteem by all who were closely associated with him, and they will miss his quiet cheerful sympathy and consideration.

He was married in Cleveland, Ohio, in 1890, and is survived by his widow, Emma (LeVake) Gifford, two sons, a married daughter, and also by his mother.

Mr. Gifford was elected an Associate Member of the American Society of Civil Engineers on October 7, 1891, and a Member on January 1, 1896.

EDLOW WINGATE HARRISON, M. Am. Soc. C. E.*

Died November 27, 1925.

Edlow Wingate Harrison was born in New York, N. Y., on May 9, 1851, the son of Samuel E. and Sarah Edlow (Williams) Harrison. He received his early education in the public schools of his native city and, later, at the New York Free Academy. In 1866 he entered the Engineering Classes of Cooper

* Memoir prepared from information supplied by Frederic Molitor and J. Waldo Smith, Members, Am. Soc. C. E.

Union from which he was graduated in 1872. During the same period he was tutored in Mechanical Engineering.

Between 1872 and 1884 he was in the office of the firm of Bacot, Post, and Camp, at Jersey City, N. J., laying the foundation of that broad detailed experience and technique that he used so successfully in his later career as Administrative and Consulting Engineer.

From 1885 to 1893 Mr. Harrison served as Engineer of the State Board of Assessors of New Jersey and as such originated the methods of physical valuation of all railroad property that are still used in taxation. He also served as Expert for the State in all the litigation establishing the legality of the tax legislation.

In 1888 he held the position of Expert Engineer for the State of New Jersey and the Pennsylvania Railroad Company in the litigation in opposition to the Baltimore and Ohio Railroad Crossing of the Arthur Kill without the consent of the State.

From 1888 to 1890 he projected, and with his associates incorporated and built, the Raritan River Railroad of which he served as Chief Engineer and Director until 1907 and as Director and Vice-President until his death.

Mr. Harrison was keenly interested in public affairs both in Jersey City and in the State of New Jersey. He was one of the first to project the Hudson County Boulevard, running from Guttenberg to Bergen Point, Bayonne, of which he was Chief Engineer from 1892 to 1898. This highway which was planned with great skill and foresight, is considered one of the greatest assets of Hudson County.

He was largely instrumental in securing an additional gravity supply of pure water for Jersey City which for many years had been supplied with the badly polluted water from the Lower Passaic River near Belleville. About 1900 a contract was made by the Jersey City Water Supply Company, of which Mr. Harrison was Chief Engineer, to supply 70 000 000 gal. daily from the Rockaway tributary of the Passaic River. This work involved an expenditure of about $7 500 000 and was completed in 1903.

Jersey City and the adjoining territory were the termini of numerous trunk-line railroads, and Mr. Harrison was called in consultation in many controversies which arose as to the proper valuation for taxation to be placed on the railroads and also on questions of valuation regarding docks and shipping. He was frequently in Court as an expert witness on valuation in connection with the diversion of water, power, and many other matters.

From 1897 to 1911 he held the position of Expert Engineer for the Central Railroad of New Jersey, the Lehigh Valley, the Erie, the Pennsylvania, the Delaware, Lackawanna and Western, and the New York Central (West Shore) Railroad Companies, as well as the North German Lloyd Steamship Company, in the examination and determination of the physical value of the terminal yards, piers, and structures on the New Jersey shore front of New York Harbor and the settlement of State and municipal taxes.

He also acted as Consulting Engineer in the examination and report as to the true present value of the Hudson River Tunnels in the settlement of taxa-

tion by the State, and made an examination and report for the City of Hoboken, N. J., on the value of lands and improvements along its water-front.

Mr. Harrison was consulted in the planning of the trunk sewer in the Passaic River Valley from Paterson, N. J., to the point of discharge in New York Harbor.

In addition to his other work, he carried on a general engineering practice, which included roads, bridges, and other important construction. He had studied the sewerage systems and road works of Great Britain, France, Germany, and Switzerland, and had used some of the foreign methods of design and construction in his work in this country.

Mr. Harrison had an attractive personality and was a man of unusual mentality, being possessed of a remarkable memory for detail, which served him well during his engagements on important cases in Court. Sanitary matters interested him, and he was influential in promoting better conditions of living. He was an engineer of remarkable qualifications with deep insight and a seeker and finder of truth.

He was interested in military matters and served in the Fourth Regiment, National Guard of New Jersey, for fifteen years. He was a member of the American Institute of Consulting Engineers, the Railroad Club of New York, the New Jersey State Sanitary Association, the New Jersey State Chamber of Commerce, the Hudson County Historical Society, the Carteret Club of Jersey City, and the Yountakah Country Club. Mr. Harrison was also a member of St. John's Protestant Episcopal Church of Jersey City, until he moved to Montclair, where he became affiliated with St. James Protestant Episcopal Church.

In 1885, he was married to Martha A. Bumsted who died the following year. In 1889, he was married to Harriet Taylor McLaughlin who died in 1910.

Mr. Harrison was elected a Member of the American Society of Civil Engineers on June 3, 1885.

KENNETH PHIPSON HAWKSLEY, M. Am. Soc. C. E.*

Died May 2, 1924.

Kenneth Phipson Hawksley was born on September 15, 1869, in London, England. He received his education at Wellington College, Sandhurst, and, subsequently, at Trinity College, Cambridge.

He was first employed under the direction of his father, the late Charles Hawksley, a Past-President of the Institution of Civil Engineers, who was a partner in the firm of T. and C. Hawksley. He also served as Apprentice in the workshops of the Glenfield Company, at Kilmarnock, Scotland, from 1889 to 1892, and in 1893 studied at University College, London.

* Memoir compiled from data on file at Society Headquarters and from information supplied by H. W. Davey, Esq., London, England.

From 1893 to 1896, Mr. Hawksley was engaged on various Parliamentary surveys and in superintending the construction of engineering work for the firm of T. and C. Hawksley, including water-works and pumping stations, of which the most important were those at Sunderland, Bristol, Coventry, and Banbury, as well as impounding reservoirs at Newcastle-on-Tyne, Bristol, Huddersfield, and other places. He was also engaged in the construction of gas-works at Darlington, Sunderland, Lowestoft, and Folkestone, and of sewerage works at Birmingham and Yeovil, as well as numerous smaller works. He continued in similar work until 1899 and took part in the responsibility of the design, as Engineering Assistant for the firm.

On January 1, 1900, Mr. Hawksley entered into partnership with his father and with him carried on a large practice as Consulting and Constructional Engineers, principally with reference to water, gas, and sewerage works. He became well known as one of the foremost experts in water supply engineering in England and was in constant request in giving evidence before Parliamentary Committees on behalf of water corporations and companies.

During the World War he was appointed Consulting Engineer for Water Supplies in connection with the Ministry of Munitions and designed and carried out the construction of large works for the Gretna, Avonmouth, Queen's Ferry, Henbury, and other large explosive plants.

At the time of his death Mr. Hawksley was a Member of the Council of the Institution of Civil Engineers, having been elected in 1921, and was the third generation of his family to serve in this capacity. This constituted a record, as there was no other instance of the kind in the history of the Institution. He was also a member of several other scientific societies.

Mr. Hawksley was elected a Member of the American Society of Civil Engineers on May 6, 1903.

HOWARD VERNON HINCKLEY, M. Am. Soc. C. E.*

Died March 24, 1926.

Howard Vernon Hinckley was born in Marstons Mills, Mass., on November 22, 1855. He was the son of Nathaniel and Ann (Judson) Hinckley.

His technical education was received at Worcester Polytechnic Institute, Worcester, Mass., from which he was graduated in 1876.

From 1877 to 1893, Mr. Hinckley was employed in the Engineering Department of the Atchison, Topeka and Santa Fé System. As Assistant Chief Engineer, which position he filled for twelve years—1881 to 1893—his duties included the designing of bridges, buildings, shop yards, track appliances, water stations, freight and passenger terminals; hydraulic problems on the Mississippi, Missouri, Kansas and Colorado Rivers; interlocking signals; inspection of pavements, sewers, and other public improvements in Chicago, Ill., Kansas City, Mo., Topeka and Wichita, Kans., and Pueblo, Colo.; and

* Memoir prepared by V. V. Long, M. Am. Soc. C. E.

passenger and freight terminals in Chicago, Kansas City, Pueblo, Topeka, Denver, Colo., and El Paso, Tex.

From 1894 to 1899, Mr. Hinckley, as a Consulting Engineer, was engaged on various irrigation, railroad, bridge, water-works, sewer, light plant, and paving improvements. In 1897, he built the Melan Arch Bridge over the Kansas River at Topeka at a cost of $150 000, and also designed four municipal water-works systems. He acted as Sewer Expert in a suit against the City of Topeka in 1898. As Consulting Engineer, in 1899, he designed reinforced concrete bridges for Des Moines, Iowa, Columbus, Ohio, and several other cities; chief among the improvements handled at that time, were the valuation of the Topeka Water-Works, and the surveying of Muskogee Townsite. From 1900 to 1903 Mr. Hinckley was United States Supervising Engineer for the Indian Territory.

Opening an office in 1904, he practiced as a Consulting Engineer for five years, in charge of various municipal improvements in Oklahoma. Two bridges were built on Platt National Park, at Sulphur, Okla., following Mr. Hinckley's designs. These designs were selected in Washington, as being better than any of those submitted by several bridge companies. Sewage and garbage disposal problems took up his time from 1910 to 1913. Since 1910, his work had been largely confined to bridge and highway construction, and for the last few years he was connected with the State Highway Bridge Department of Oklahoma.

Always interested in Society affairs, Mr. Hinckley was a leader in all matters of interest to Oklahoma engineers, and for years his untiring work as Secretary of the Oklahoma Section of the Society, kept Oklahoma engineers together. He also served as President of the Oklahoma Section in 1919.

Mr. Hinckley was a man of strong opinions and principles and could always be depended upon to fight for what he thought was right. He was respected and honored by all engineers who knew him and his death leaves a vacancy in the Oklahoma Section that will be difficult to fill.

On November 17, 1880, he was married to Lettie S. Cooldaugh, of Towanda, Pa., who, with their two children, Mrs. Hila H. Wunderlich, of Oklahoma City, and Charles M. Hinckley, of Ponca City, Okla., survives him.

Mr. Hinckley was elected a Member of the American Society of Civil Engineers on December 5, 1883.

JOSEPH WARREN HOOVER, M. Am. Soc. C. E.*

DIED JUNE 18, 1925.

Joseph Warren Hoover was born on November 19, 1850, on a farm about five miles north of Canton, Ohio. His parents were Daniel and Mary (Kryder) Hoover. With his two brothers, William H. and Frank K., his boyhood was spent among surroundings typical of what might be called the second stage

* Memoir prepared by H. P. Treadway, M. Am. Soc. C. E.

of pioneer development, newly cleared land, rail fences, neighborhood flour mills, brick kilns, tanneries, etc.

His education began with the country school at the nearest crossroads and was carried on at Mount Union College, Mount Union, Ohio.

A combination of circumstances determined Mr. Hoover's life work, the first incident occurring on a sweltering day when his breath was knocked out of him by the handle of a plow which he was guiding. The result was a definite rejection of farming as a life occupation. The fact that his father, Daniel Hoover, through self-training, had become a competent Surveyor and frequently surveyed the farms of his neighbors for drainage purposes, etc., probably aroused his interest in surveying. At this time a new railroad called the Valley Railroad was being run south from Akron, Ohio, passing about three miles to the west of the old farm. This may have suggested to Mr. Hoover employment other than farm work. In any event, he secured work during the summers on this new line which gave him experience in railroad surveying.

This experience doubtless led to his entering the University of Michigan in the fall of 1872. He was graduated from the Engineering Department three years later, in the spring of 1875. In this period he covered four years of work in Civil Engineering and of the three working years nearly one-half of each was spent, during the construction season, in field work for the Valley Railway Company. Of this work and character at that time, P. H. Dudley, Chief Engineer of the Valley Railway Company, wrote on September 21, 1874, as follows, "His ability, fidelity, and strict integrity are such that they commend him to any one".

After his graduation, Mr. Hoover's first professional employment was as Assistant in an Observatory in Cincinnati, Ohio. This work was very interesting, but not sufficiently lucrative, and in the fall of 1875 he secured employment as Chief Engineer of the Indianapolis Bridge Company. He invested in the Company all the money he had saved from his summer employment in previous years. Within a year, however, the Bridge Company encountered financial difficulties, and Mr. Hoover lost the investment he had made and his employment as well. The effect of this experience was to implant in him a deep-seated and permanent conservatism which resulted in a life of hard work, careful planning, and avoidance, as far as possible, of large hazards.

He returned to Canton and for a time was engaged in designing and building highway bridges. In 1878, he was employed as Engineer by the Wrought Iron Bridge Company, of Canton. He found this work interesting, sufficiently remunerative, and well appreciated, but much too confining, and, in 1884, having made a contract with the Wrought Iron Bridge Company to represent it as General Western Agent, Mr. Hoover moved with his family to Kansas City, Mo.

His contract was such that he was justified in devoting himself to building up a creditable business, which he did through the succeeding years. The facilities of the Company were adapted only to the fabrication of highway bridges and steel viaducts and his field of operations, therefore, followed that line.

In the West at that time there were few engineers engaged in the design of such work. Most of the county engineers were only surveyors and not trained for economical bridge or structural design, and Mr. Hoover's services were often sought by engineers for cities, counties, street railway companies, etc. In the face of severe competition, he built up this business largely through his ability as a designing engineer. He was particularly in demand in the case of structures which were out of the ordinary and which could be properly analyzed and designed only by a man of thorough training and experience.

As time passed his business became more and more that of a builder and less that of an engineer, and his later success was based on two characteristics, tireless attention even to detail and absolute faithfulness to the interests of his customers. These deep-seated characteristics made possible his success in a field marked with the wrecks of many failures.

Mr. Hoover operated continuously under his contract with the Wrought Iron Bridge Company for sixteen years until 1900, when that Company, with about twenty-five other shops, was merged into and became part of the American Bridge Company.

In the next two years, 1901 and 1902, the American Bridge Company passed through a period of re-organization of its various plants and Mr. Hoover continued as its General Agent in a limited territory. That temporary association, however, was not to the best advantage of either party and, in 1902, he became the General Western Agent for the Canton Bridge Company, of Canton.

At some time in 1904 or 1905 he sustained a physical injury which resulted about 1906 in the loss of his sight. After continuing for some time with the hope of recovery, he retired in 1909 as General Agent of the Canton Bridge Company. For a few years more he gave active attention to certain enterprises in which he was interested. As soon as possible, however, he put his affairs in order. During the remainder of his life he attended to his investments and with the aid of his family and others devoted much time to the reading of history and current events, and to music.

He tolerated motor cars when necessary, but preferred his one-horse carriage until it became the single survivor of its kind in Kansas City. He was a Mason, York and Scottish Rite, and a Shriner, a member of the University Club, and of the Unitarian Church, of Kansas City.

Mr. Hoover's strongest characteristic, perhaps, was accuracy. He was intolerant of careless error, of negligence, and of duplicity, but patient with failure where there had been genuine effort. He was faithful to all trusts placed in him. His word was as good as any bond, and he held the complete confidence of all who really knew him. Although naturally cautious, or perhaps because he was cautious and conservative in his general plan of life, he was a good loser in the case of any unexpected, unforeseen disaster. Such occurrences, after analysis and diagnosis, were simply "spilled milk".

His life was one of faithful service to others. It was successful in the matter of financial reward, but the loss of his sight deprived him of the real fruits of his many years of effort.

Mr. Hoover was married on December 30, 1875, to Mary Catherine Ruthrauff, of Stark County, Ohio, who, with a daughter, Helen Hoover Secrest, and a son, Frederick R. Hoover, Assoc. M. Am. Soc. C. E., survives him.

Mr. Hoover was elected a Member of the American Society of Civil Engineers on June 6, 1888.

OBERLIN SMITH, M. Am. Soc. C. E.*

Died July 18, 1926.

Oberlin Smith, the eldest son of George R. and Salome Kemp Smith, was born in Cincinnati, Ohio, on March 22, 1840. His father was of English parentage, the ancestral home having been at Stoke, Dorsetshire, England, where Mr. Smith's grandfather operated a flour mill and dealt largely in wheat and other grains. The Napoleonic Wars brought heavy losses to the English people and were doubtless responsible for the migration of the family to America between 1830 and 1840.

Oberlin Smith's parents owned and lived on two or three Ohio farms during his early years. The boy, however, had a bent for mechanics and when only fifteen years of age, he built a steam engine designed entirely from descriptions given him by his father. He also constructed toy water-wheels, pumps, and other apparatus, finally attempting a cylinder electrical machine made from bottles of various sizes with holes cut in the bottom with a sharp file. He made frequent visits to neighboring sawmills and on his return built engines which produced successful results.

About 1857 his father died and his mother, with her three children, removed to Bridgeton, N. J., where relatives were living. Notwithstanding his youth, Oberlin Smith took his father's place, assisting his mother in superintending the education of his brother and sister. He attended the West Jersey Academy and studied engineering at the Polytechnic Institute, Philadelphia, Pa.

While quite young, he entered the machine shop of the Cumberland Nail and Iron Company of Bridgeton and his exceptional mechanical ability was soon recognized. While working in the pipe-mill attached to the plant, he noticed that when the ends of pipe were cut off, it took the time of an additional man to remove the burr on the inside edge caused by the cutting process. He thereupon invented a simple attachment which enabled the operator to cut and ream the pipe at the same operation. The value of this labor-saving device was so apparent, that the Superintendent gave him $50. Mr. Smith frequently declared that no reward subsequently received for his inventions was so gratifying as this unexpected "bonus".

In 1863, he went into business for himself, building a small shop and conducting a general trade in gas and steam-fitting, plumbing, and architectural iron work, such as iron fences, verandas, etc. He also engaged in a general jobbing and repair business, followed by the making of a few sizes of foot-

* Memoir prepared from information on file at the Headquarters of the Society.

presses for canning factories, a beginning which subsequently led to the specialty of presses and dies as an almost exclusive product.

On January 1, 1864, Mr. J. Burkitt Webb (afterward a Professor at Cornell University and Stevens Institute of Technology) became a partner, under the firm name of Smith and Webb. Mr. Webb was also noted for his originality and with Mr. Smith frequently startled their fellow townsmen. For instance, they built the first automobile ever seen in Southern New Jersey. The machine consisted of an old buggy carrying a small boiler and engine that drove the rear axle by means of sprockets. The contrivance was crude, the steam valve, for instance, being operated by a loose socket wrench instead of a hand-wheel. The car traversed the streets of Bridgeton to the delight of the inhabitants, but came to grief when descending a hill, a jolt causing the socket wrench to fall into the street, and there being no brake, it was impossible to stop. The result was disastrous to the car, but fortunately the occupants were not seriously hurt.

The firm of Smith and Webb was dissolved after several years and Mr. Frederick F. Smith, a younger brother, became a partner under the name of Oberlin Smith and Brother. In 1873, the shop and a part of the tools were sold and new buildings were erected in East Bridgeton, N. J., and on January 1, 1877, the Ferracute Machine Company was organized, with Mr. Smith as President, the regular products of presses and dies being supplemented by a number of special automatic machines for the working of metals. These machines were largely abandoned later, in order to concentrate on the development and improvement of presses with their dies and other legitimate attachments.

On September 27, 1903, the buildings of the Company were completely destroyed by fire, together with most of their contents. Plans for new and improved shops were immediately prepared and erection was begun. The new buildings are fireproof and fitted with modern devices, many of them designed by Mr. Smith, all the construction having been subject to his oversight and superintendence which was performed with the energy of a much younger man.

Mr. Smith made several European tours of engineering observation, and, in addition to his wide acquaintance among American engineers, he knew the leading mechanical engineers of Germany and England. He was a frequent contributor to the technical press and, in 1896, published a book entitled "Press-Working of Metals", which had a wide sale. This book covers a field that had been but little exploited to that time.

He had taken out about seventy-five patents for inventions a number of which are connected with the manufacture of presses and other machines for cutting and forming metals. His inventive genius, however, covered a wide field as testified by patents on improvements in automatic egg boilers, pill machines, milk-shakers, a system of gearing for turbines of ocean steamers, etc.

In common with other self-made men, Mr. Smith possessed an unusual amount of perseverance and determination. Nothing gave him so much pleasure as to attack hard problems, especially if his competitors had failed in their solution. In his younger days he started his daily task at an early hour and frequently worked long into the night. In recent years he had

become more careful of his health and allowed nothing to interfere with his daily automobile rides. He always took an interest in the mechanical features of the better makes of automobiles of which he had several. Among his inventions is an electrical device by which the pressure of a button at a considerable distance from his garage caused the doors to automatically open and be ready for the car when it arrived.

He was one of the most approachable of men and seemed to enjoy being interrupted, having had the rare faculty of immediately resuming his previous occupation at the point where he left it on the arrival of the visitor.

Mr. Smith served the State of New Jersey as Commissioner to the Pan-American Exposition in 1901 and as a member of the New Jersey Department of Conservation and Development. He was also a Director of the Cumberland National Bank; a member of the Bridgeton Commercial League; a Past-President of the American Society of Mechanical Engineers, and a member of the American Institute of Mining and Metallurgical Engineers; American Institute of Electrical Engineers; American Iron and Steel Institute; American Academy of Political and Social Science; Society for Encouragement of Arts, Manufactures, and Commerce; Franklin Institute, Art Club, and Engineers Club, of Philadelphia, Pa.; Lotus Club and Engineers Club, of New York, N. Y.; Cohanzick Country Club, Bridgeton, and other organizations.

On December 25, 1876, he was married to Charlotte Hill of Bernardston, Mass., who died in 1918. He is survived by his son, Percival N. Smith, a daughter, Madame Velko Roditchevitch, a brother, Fred F. Smith, and a sister, Emily Smith.

Mr. Smith was elected a Member of the American Society of Civil Engineers on September 3, 1884.

ROBERT SOMERVILLE, M. Am. Soc. C. E.*

Died October 19, 1925.

Robert Somerville was born on May 24, 1851, in Richmond, Va., and was reared there, living on Franklin Street, near where the Jefferson Hotel now stands. His early education was received largely at the University School of Richmond, a private school, of which the Assistant Principal was Thomas R. Price, Jr., who was called from this school to be Professor in Mathematics at the University of Virginia and, later, at Columbia University in New York, N. Y. It was the thorough instruction received by Mr. Somerville from this eminent mathematician which lent so much to his success as an engineer.

After the Civil War Mr. Somerville attended the University of Virginia, where he became a member of the Phi Kappa Psi Fraternity, to which Chapter, a few years later, President Wilson was elected. Mr. Somerville did not take any degree at the University of Virginia, but completed the resident work

* Memoir prepared by C. H. West and W. E. Elam, Members, Am. Soc. C. E.

necessary for a Civil Engineer and later would have been entitled to the degree had he applied for it.

On leaving the University of Virginia he was employed by the Chesapeake and Ohio Railroad Company and worked under Maj. Peyton Randolph for several years in the vicinity of Huntington, W. Va. The lack of home life and social advantages in that country caused Mr. Somerville to abandon his profession for a time and he returned to Richmond and formed a partnership with his father in the grain and commission business, under the firm name of R. B. Somerville & Son. He did the bookkeeping and kept all the records for the firm, and his father did all the buying and selling.

While in Richmond and in this business he was married to Mary Lee, a close relative of Gen. Robert E. Lee. They had long been sweethearts and the marriage had been delayed by her ill-health. When she died a year later, Mr. Somerville left Richmond and again entered the engineering field, this time to stay for the remainder of his life.

He was again employed by the Chesapeake and Ohio Railroad Company on the reconnaissance and location surveys for the extension from Huntington to Cincinnati, Ohio. It was while doing this work in Kentucky that he was offered a position with the Board of Mississippi Levee Commissioners. He accepted this offer and came to Greenville, Miss., where he lived until his death on October 19, 1925. With the exception of a short employment with the Georgia Pacific Railway Company (now the Columbus and Greenville Railway Company) on the location of its lines through the Mississippi Delta, Mr. Somerville was steadily employed by the Levee Board for the remainder of his life, and it was in this position that he did his real life work. He was employed as an Assistant Engineer for many years and, finally, the position of Assistant Chief Engineer was created, to which he was promoted. His principal work which will live after him was the keeping of records that show all the work of the Levee Board.

Mr. Somerville was always careful, conscientious, and conservative in all matters coming under his care; and his passion for record keeping gave the Mississippi Levee District the most complete records of any Levee District as far as the writer knows. In passing, it might be added that a few years ago the writer tried to collect data from all the Levee Districts along the Mississippi River, similar to the information contained in the records of the Mississippi Levee District, and had to abandon it because so many districts could only supply the total expenditures and could not separate the cost of construction from the other expenses such as overhead, interest, etc. The desired information was what had been spent on actual construction, and in the other districts it could only be estimated. Yet Mr. Somerville, at a time when cost accounting was practically unknown, devised a system of record keeping which was complete enough to supply reliable data when legislation was being urged in Congress for the improvement of the Mississippi River.

Mr. Somerville had a mild and most gentlemanly manner, coupled with firmness when necessary. His devotion to duty will ever be a landmark in the community.

He was prominent in church and fraternal circles. For at least forty years he was an active member of the Board of Stewards of the First Methodist Episcopal Church of Greenville, and he served the church in practically every capacity possible for a layman. In each and every place he was ready, competent, and willing to serve at all times. He was also quite active in Masonic circles, having been a member of the Blue Lodge, Chapter, Council, Commandery, Shrine, and Eastern Star. He served as the highest official of each of these locally, and held many other local offices in the bodies; he also served the Grand Chapter, Royal Arch Masons, as Grand High Priest for the year 1913.

On September 16, 1885, Mr. Somerville was married to Nellie Nugent, daughter of Col. William L. Nugent, formerly of Greenville, but then residing in Jackson, Miss. To this union there were born four children, Robert N., Abe D., Eleanor (Mrs. A. W. Shands), and Lucy R., all of whom survive him. He will be missed by his devoted family and those who came in contact with him in a social and professional way.

Mr. Somerville was elected a Member of the American Society of Civil Engineers on June 1, 1887.

REUBEN JOSEPH WOOD, M. Am. Soc. C. E.*

Died October 15, 1925.

Reuben Joseph Wood was born at Kansas City, Mo., on June 19, 1884. He was the only child of William and Josephine Wood. He received his early education in the public schools of San Francisco, Calif., which city had been his home since 1885. In 1903, he entered the University of California and was graduated with the degree of Bachelor of Science in Civil Engineering in 1907.

After his graduation Mr. Wood was employed for about two years with the Southern Pacific Railroad Company, designing steel and reinforced concrete structures. Following this engagement he entered the contracting business, specializing in building work.

In 1911, he joined the Staff of the City Engineer of San Francisco as an Assistant Engineer. From this position he advanced to that of Consulting Structural Engineer, in which capacity he served until his death.

During his employment in the City Engineer's Office, Mr. Wood was intimately connected with the design of the structural elements of many important public improvements, such as the high-pressure fire system, which included some large reinforced concrete tanks and a great number of cisterns; the Municipal Railway with its reinforced concrete car barns and trolley poles; the Stockton Street and Twin Peaks Tunnels and Stations; the Hetch Hetchy Project, including the Moccasin Power House and the steel towers on the transmission line, and the O'Shaughnessy, Priest, Lake Eleanor, and Early Intake Dams.

* Memoir prepared by Nelson A. Eckart and L. H. Nishkian, Members, Am. Soc. C. E.

Although never of robust health, Mr. Wood possessed to a high degree those qualities of energy, perseverance, resourcefulness, and loyalty which won recognition for him in the engineering service of the City. Intensely interested in civil engineering, he found time to acquire a large store of accurate information in the broader field of life. The quality of his work as an engineer, the ability he displayed, and his steadfast devotion to his profession gave promise of a useful and honorable career had his life been prolonged.

He was unmarried and made his home with his mother, by whom he is survived.

Mr. Wood was elected a Member of the American Society of Civil Engineers on August 31, 1925.

THOMAS PATRICK O'SHAUGHNESSY, Assoc. M. Am. Soc. C. E.*

Died July 11, 1924.

Thomas Patrick O'Shaughnessy was born in San Francisco, Calif., on July 21, 1893, the son of Michael and Anna E. O'Shaughnessy. He was the eldest of four children and was educated in San Francisco and Oakland, Calif., receiving the degree of Bachelor of Science from St. Mary's College, in Oakland, in 1915.

Following his graduation Mr. O'Shaughnessy worked as Assistant Engineer for the California Highway Commission until November, 1917, when he entered the U. S. Army as a Cadet Pilot in the Air Service of the Signal Corps. He was in this service for more than a year, after which he was engaged with the Bureau of Public Roads of the U. S. Department of Agriculture until July, 1919. He then re-entered the service of the California Highway Commission, as Assistant Engineer of Construction.

In July, 1921, Mr. O'Shaughnessy became Concrete Inspector on the Hetch Hetchy Dam for the City of San Francisco, in charge of the pouring of precast work and, later, in charge of the night pouring at the dam. After almost a year in this position he was made Resident Engineer for the Tule and Baxter Creek Irrigation Districts in Lassen County.

This memoir must of necessity be brief, for the life of "Tom" O'Shaughnessy was a short one, his death occurring a few days before his thirty-first birthday. During his few years of active work he made countless friends, with whom he has left ineradicable memories of a dauntless spirit of youth coupled with a keen sense of responsibility and judgment, which made it possible to employ him in positions of trust. His was a valiant, high-hearted nature, and it is hard to think that he is no more a part of the actual life of his associates, that henceforward he will be only a happy memory. His friends are happy to have loved him, happy that he has been happy with them, happy to have been "a port where he hath fitted himself for what sea he saileth to."

* Memoir prepared by a Committee of the San Francisco Section, Am. Soc. C. E.

One of his fellow engineers on the Hetch-Hetchy work writes of him:

"To those who knew him best, Tom's dominant characteristic was a warm heart coupled with a fearless and independent mental attitude which was an inspiration to his associates. He was a fine fellow and I loved him."

He had a "way with him" which endeared him to all, old and young, and, because his work was for the most part in mountainous country where Nature is at its strongest and most beautiful, his associates will look for him in all that is most beautiful and strong:

"In the sunlight of a morning in June, in the soft green grass that carpets the uplands in May, in the sturdy oak whose head is unbowed by the winds of a hundred winters, in the mellowed radiance of the harvest moon, in all that we love in Nature we will look for our departed friend."

Mr. O'Shaughnessy was elected an Associate Member of the American Society of Civil Engineers on May 28, 1923.